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A Study of the Soil Conditions In the Edmonton Area

W.E. Jubien
1956

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SYNOPSIS

A large body of information, in the form of various test results, on the soil from the Edmonton area has been collected by others but it has never been fully analysed. In this thesis the results of these tests are studied and an attempt is made to analyse the data to obtain general relationships between the various soil properties. The investigation is divided into six main parts in each of which a different property is investigated. The six divisions are plasticity characteristics, unconfined compressive strength, consolidation characteristics, swelling pressure, modulus of deformation in confined compression and blow count on the sampling spoon.

Two types of plastic soils are distinguished on the basis of plasticity characteristics. Relationships between the unconfined compressive strength and moisture content, and between blow count on the sampling spoon and unconfined compressive strength are suggested. Possible methods for determining the swelling pressure from the liquidity index and for calculating the slope of the virgin branch of the consolidation curve are also indicated but these procedures require further research to establish their validity.

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THE UNIVERSITY OF ALBERTA

A STUDY OF SOIL CONDITIONS IN THE EDMONTON AREA

A DISSERTATION
SUBMITTED TO THE SCHOOL OF GRADUATE STUDIES
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OF MASTER OF SCIENCE

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by

WILLIAM E. JUBIEN
EDMONTON, ALBERTA

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A STUDY OF THE SOIL CONDITIONS IN THE EDMONTON AREA

I INTRODUCTION

During the past twenty years the properties of the soils in the Edmonton area have been studied from the point of view of soil mechanics. This study has involved laboratory testing to determine the properties of the soil which the theories of soil mechanics have shown to be significant and application of the results of the tests to the design of the foundations. For each site to be investigated, a program of field sampling and laboratory testing is laid out and recommendations for the design of the foundations are made on the basis of the test results. Until the present time the data obtained as the result of the individual foundation investigations has never been brought together and analyzed in an attempt to draw general conclusions about the properties of the soils in the Edmonton area. The purpose of this thesis is to report the results of an analysis made for the purpose of drawing general conclusions about the significant properties of Edmonton soils, from the point of view of soil mechanics.

For the most part, the data used in its preparation has been selected from foundation reports prepared within the past five years. This particular data was used both because it was the most readily available and because the techniques used in sampling and testing the soil have been reasonably constant throughout the period. The data was analyzed by plotting various soil properties and by constructing histograms to show the frequency of occurrence of certain test properties. Most of

the correlations attempted were suggested by results previously published by various authors, but some were prompted by the general nature of the data suggesting certain general relationships that might exist between two or more soil properties. Throughout the work, one of the primary objects has been to produce correlations that could be used to predict the results of one test from another simpler or less expensive test.

From the beginning it was known that because the data to be used was originally obtained for use in preparing individual foundation reports it would possess certain limitations for use in the analysis. For example, over 70% of the consolidation results were for soils sampled from five to six feet below the surface and therefore having nearly the same effective overburden pressure. This makes an analysis of the effect of overburden pressure on the consolidation characteristics impossible. However, it was felt that the large amount of data available provided an excellent opportunity of obtaining a correlation between the soil properties in spite of minor shortcomings.

In some instances, the result of the analysis has been to indicate certain relationships without either proving or disproving them. Special investigations are required to check the validity of these suggested relationships.

In other cases the analysis has resulted in useful correlations between various properties of the soils found in the Edmonton area.

II LOCATION AND TOPOGRAPHY

The City of Edmonton is located near the western edge of the great plains region of Canada. In the general Edmonton area, the ground

surface is gently rolling. It shows frequent evidence of glaciation which has formed many of the main topographic features of the region. Both immediately east and west of the City the topographic features have been produced by moraine deposits. As a result, the surface is rough and erratic, with frequent low spots which hold water during the spring. Within the City limits, the surface soils are generally post-glacial lake bed deposits and the ground surface is therefore quite level. Within the City the outstanding topographic feature is the valley of the North Saskatchewan River, which is 165 feet below the level of the plain .

In general, there is very little development in the river valley itself, most of the buildings in the City are on the old lake bed above the river valley. For this reason most of the test results used in this analysis were obtained from soil sampled on the flat areas well above the river and the results of the analysis do not apply to the recent river valley deposits.

III GEOLOGY

1. General -

The rocks in Alberta, with the exception of the Pre-Cambrian metamorphic rocks in the northeast, are of sedimentary origin. These sediments range in age from ancient Pre-Cambrian to recent Tertiary deposits. They were not laid down continuously but periods of sedimentation were followed by other periods in which the land was exposed to erosive forces.

Beneath the plains section the sedimentary rocks are formed in a shallow syncline, with its longitudinal axis generally north and south.

Proceeding towards the west, progressively younger formations are encountered in the plains region.

In the vicinity of Edmonton, the upper rocks consist of interbedded sandstones, shales and some coal seams of Cretaceous age. This formation, known as the Edmonton formation, extends from north of Edmonton into the southern part of the province. Within the immediate vicinity of the City it is buried under soil deposits except in the river valley where the soil has been removed by the river.

2. Soils -

The soils in the Edmonton district are composed of sediments of Pleistocene and Recent series. They are described in the table of formations, Plate 1, and the description of formations which follows. Plate 1 and the information for the description of most of the formation was taken from Collins and Swan (Ref. 5). The descriptions given in this reference for the St. Ann area west of Edmonton were generally confirmed by Duff (Ref. 8), in his investigation of the soils in the Edmonton area.

3. Description of Formations -

a) Saskatchewan Gravels and Sands -

These gravels and sands are quite widespread over the province. They are exposed in deep sections along the North Saskatchewan River valley. According to Rutherford (Ref. 14), in the district immediately adjacent to Edmonton along the river valley to the west of the City, the sands are the more prevalent whereas to the east of the City and in several places somewhat removed from the City, gravels are more prevalent.

The coarse materials are composed of undecomposed bedrock of local origin and of pebbles and boulders of Cordilleran origin.

b) Gray Till -

This is the oldest till in the area and is generally found overlying the Saskatchewan sands and gravels. It is not continuous but is found only in low places. Thus it appears that the deposition of the gray till was followed by a long period of erosion. The remaining till layer is quite thin and can usually be identified by its gray colour. The composition of the till shows that it was transported by ice moving into the area from the east.

c) Lower Interglacial Deposits -

Above the gray till in some places is a sand layer that separates the gray till below from the brown till above. The sands show great variation as they are local accumulations of interglacial deposits.

d) Brown Till -

Concerning the brown till, Duff (Ref. 8), states "This till overlies the lower interglacial beds and the gray till and is quite thick, sections up to 33 feet having been observed. It is massive, very stony and boulders up to 25 inches in diameter were observed. When dry it has a light brown colour and weathers to a columnar wall. When wet or on a fresh surface, the till is sticky and pliable."

e) Upper Interglacial Stage -

Deposits assigned to this stage are silts, clays, sands, and sand and gravel deposits. This stage indicates a time break between the brown till below and the silt till above.

f) Silt Till -

There is some doubt as to whether this material is a true glacial till deposit or whether it was laid down as a lakebed deposit in a shallow lake formed during the retreat of the glacier. In any event, the silt till is less plastic than the brown till below and generally contains fewer pebbles than the brown till.

4. Summary -

From the engineering viewpoint, the main feature indicated by geologic evidence is that the soils in the Edmonton area are of glacial or interglacial origin. This means that except for the surface soils which may have been laid down since the last glaciation, all of the soils in the area have been subjected to the weight of a large thickness of ice so that no soft strata can exist at great depth and cause long-term settlement under building loads. For this reason, for buildings, foundation investigation much deeper than the proposed foundation level is not generally required.

Since it is known from geologic evidence that all the soils are either of glacial or interglacial origin, it can be expected that foundation conditions will be very irregular due to the erratic nature of the deposits generally associated with glaciation. This indicates that detailed investigation of each site will be required in order to assess the foundation conditions. However, certain different soil types can be distinguished on the basis of geological knowledge and it may be possible to identify these same soils from their engineering properties. During the preparation of this thesis, an attempt has been made to correlate the

soil description made by the drilling foreman with the geologist's description of the soils encountered in the area. This attempt has not been successful, probably because the test holes investigated were generally shallow and did not show a sufficiently deep cross-section of the material to show the complete series of formations so they could be distinguished from each other.

The results of the analysis of the engineering properties does indicate that two different types of soil are frequently encountered. The more highly plastic of these types is probably the material known to the geologist as the brown till. The other material may be part of the upper interglacial stage or the silt till or a combination of the two. Co-operative investigation by a soils engineer and a geologist would be required for a definite statement on this subject.

IV DATA AVAILABLE

Before describing the methods of analysis used to investigate the foundation conditions in the City of Edmonton, it is necessary to give a brief description of the field sampling methods that were used and record the types of test for which data was available.

1. Field Investigation -

From geological evidence it is known that all the soils except the ones at the surface have been preloaded by the weight of at least one glacier. For this reason the foundation investigation for a building is not concerned with a search for possible soft or highly compressible soil strata at great depths, but with the specific properties of the soil, from the surface to a depth of twenty or thirty feet, on which the building will be founded.

The usual method of obtaining this information consists of putting down one, two, or more auger holes to the required depth at the site of the building and performing various field and laboratory tests on the soils encountered. As the hole is advanced the soil is visually classified and at the same time both disturbed and undisturbed samples are obtained. The disturbed samples, which are taken at approximately two foot intervals, are used for obtaining the moisture content profile in the test hole. In cohesive soils, undisturbed samples are obtained, in three inch diameter Shelby tubes, at approximately five foot intervals. The Shelby tubes are driven into the soil by a 30-pound weight dropped 30 inches. The number of blows of this hammer required to produce various penetrations of the sampling spoon is recorded in the field boring log.

When undisturbed samples of cohesionless soils cannot be obtained, some field test such as the standard penetration test described by Terzaghi, may be used to determine the density of the material in place. In addition to other data recorded in the field log, the position of the water table is noted, if possible.

2. Laboratory Tests -

The following laboratory tests are usually run on the samples obtained: -

i. Moisture content; expressed as a percent of the dry weight of the soil.

ii. Liquid and plastic limits (ASTM D423-39 and D424-39).

iii. Unconfined compression strength - see procedure in Appendix B.

iv. Combined consolidation and swelling pressure test - see procedure in Appendix B.

V FOUNDATION CONDITIONS

In general the foundation conditions are good. The prevailing soils are glacial tills with liquid limits between 30 and 85. Interspersed in the till in an apparently random manner are deposits of silt, fine to coarse sand and gravel. The erratic nature of the deposits makes thorough foundation investigation necessary in order to establish the foundation conditions for each building because buildings quite close together may have very different foundation conditions.

One common characteristic of many soil samples taken near the surface is a granular or nugget structure. It is thought that this structure was formed due to shrinkage of the soil on drying. Subsequent wetting does not appear to destroy the structure. Nugget structure can be readily identified by breaking a lump of soil in the hand and noting the characteristic granular structure. In some locations nugget structure extends to depths of ten feet while in others it is not found below a depth of two feet. One consequence of this granular structure is an increase in the permeability of the till caused by the ability of water to seep through the hairline cracks between the soil grains. Another result of the desiccation is to lower the compressibility of the soil on subsequent loading because the soil has previously been consolidation by the large surface tension forces which develop on drying.

Any soil that was laid down before the last glaciation has been consolidated under the weight of at least one glaciation and most of the more recent deposits have been consolidated due to desiccation and therefore settlement problems do not arise in connection with buildings of average size and weight although, for heavy loads, settlement due to elastic compression rather than consolidation may be encountered.

Since settlement of buildings is not usually a problem, the governing factor in the design of foundations is usually soil strength. In general the soils found in the area have strengths ranging from medium to very high. Histograms have been prepared to show the distribution of the results of unconfined compression tests on the Edmonton soils. Plate 2 a shows the distribution of strength for all of the soils tested and plates 2 (b) to 4 show the strength distribution for soils grouped according to liquid limit. From these diagrams it can be seen that none of the samples tested showed a strength of less than 0.5 tons per sq ft and over 60% of the test results were between 1.0 and 2.5 tons per sq ft. While the diagrams were being prepared it was noticed that most of the soils with a strength of less than 1.0 tons per sq ft had a notation such as sandy or gravel. Unconfined compressive strengths for these soils do not give a true picture of their ability to support loads.

Although consolidation is seldom a problem and the soil is generally strong enough to support reasonable foundation pressures without shearing, damaging foundation movements have occurred due to swelling of the soil. The highly preconsolidated glacial tills in the area undergo considerable volume change with changing moisture content. Enough pressure is developed by these soils when they swell to break foundation slabs in contact with the ground or to lift the foundations for light buildings such as houses. In order to keep changes in moisture content to a minimum, weeping tile drains are often installed at footing level and concrete slabs in contact with the ground are reinforced to prevent them from cracking due to movements caused by swelling of the soil.

In order to be able to predict the magnitude of the swelling pressure that a soil may exert a swelling test has been devised. This test is

run in conjunction with the consolidation test using the procedure outlined in the appendix.

VI METHOD OF PRESENTATION OF RESULTS

The analyses made can be divided into six categories, based on the principle tests under investigation. They are : -

i. Atterberg Limits and Natural Moisture Content;

ii. Unconfined Compressive Strength;

iii. Consolidation Characteristics;

iv. Swelling Pressures;

v. Modulus of Deformation in Confined Compression;

vi. Blow Count on the Sampling Spoon.

Each investigation will be described and discussed separately under the headings : -

i. Items investigated for each group;

ii. Results of each investigation with a discussion of its implications;

iii. Summary of the results of the analyses.

VII ATTERBERG LIMITS AND NATURAL MOISTURE CONTENT

The following items were investigated: -

i. Plasticity chart;

ii. Frequency of distribution of liquid limits;

iii. Frequency distribution of liquid limits within six feet of the surface;

iv. Relationship between natural moisture content and the liquid and plastic limits.

1. Plasticity Chart -

The first step used in analyzing the data was to plot the results of the Atterberg Limit Tests on a plasticity chart. To make the plotted points readily available for future reference, each one was numbered as it was plotted. On a separate sheet a record was kept of the numbers, along with the location from which each test was taken. To avoid overcrowding of the plasticity chart, the data was arbitrarily divided into two approximately equal divisions and plotted on two separate plasticity charts. The two plasticity charts are shown in plates 5 and 6 at a somewhat reduced size.

Both plasticity charts show the same results. Nearly all the points plot above the "A" line and according to the Casagrande system of classification (Ref. 3), they are nearly all clays. Since the liquid limits in general range from 30 to 95, the clays would be classified as having medium to high plasticity. Using the symbols suggested by Casagrande, they would be referred to as CI and CH. Furthermore, the limit values fall within a fairly narrow band parallel to the "A" line on the plasticity chart. This shows the soils are all of the same geologic origin. This result confirms geologic information which also indicates that all of the soils in the area have a similar origin.

It was considered desirable to divide the plasticity chart into zones so that further comparison could be made of the properties of clays from the same region on the plasticity chart. For this purpose a line to be known as the "B" line was drawn roughly parallel to the "A" line. The position of the "B" line was picked to roughly divide the plotted points into two equal portions; that is, the "B" line divides the plotted points so approximately half are above it and half are below it.

Because the soils with a liquid limit greater than 70 generally plotted farther from the "A" line than those with lower plasticity, the "B" line is not a straight line but has a kink between liquid limits of 60 and 70 to keep the number of points on each side approximately the same.

The regions above and below the "A" line were each divided into smaller zones on the basis of liquid limit. The boundaries between the zones are liquid limits 25-40, 40-50, 50-60, 60-70 and greater than 70. Each zone was given a letter for identification. Plate 7 shows the manner in which the plasticity chart was divided by the "A" and "B" lines and the lettering system used to identify the various zones.

The few soils that plotted below the "A" line on the plasticity chart were set aside and no attempt was made to use them in any other analyses.

From the two plasticity charts it is difficult to see any natural groups except the band parallel to the "A" line, into which the soils fall. In an effort to find other natural groupings of the soils, a histogram of the liquid limit values was prepared.

2. Frequency of Occurrence of Liquid Limit Values -

The histogram of the results of liquid limit tests is shown in plate 8a. This histogram reveals that on the basis of liquid limits there are two types of soil in the region. The most common type of soil has a mean liquid limit between sixty and sixty-five. According to the Casagrande system of classification, it is an inorganic clay of high plasticity (CH). The other soil is an inorganic clay of intermediate plasticity (CI); its average liquid limit is between 35 and 40.

3. Frequency of Occurrence of Liquid Limits Within Six Feet of the Surface -

A histogram was also prepared (plate 8b) to show the percentage of the total liquid limit results in any range that occurred within six feet of the surface. The purpose of this was to see whether the two soil types identified on the basis of liquid limit could also be identified on the basis of the depth below the surface from which the soils had been sampled. Originally it was felt that there was a tendency for soils of high plasticity to occur nearer the surface. If such were the case the histogram shown in plate 8b would be higher on the right side than on the left. It does show some tendency to rise at the high liquid limits but when it is pointed out that the columns showing 80% and 100% of their results within six feet of the surface contain only ten and one test results respectively, it will be realized that in general the results do not indicate that the more highly plastic soil is found close to the surface.

4. Relationship Between the Natural Moisture Content and the Atterberg Limits -

A histogram (plate 9a) showing the natural moisture content for all of the soils was prepared. Its two peaks indicate that two soils, having different natural moisture contents, occur in the area. Since the

liquid limit tests indicated the same result it was decided to prepare additional histograms of natural moisture content, for soils in the following ranges of liquid limits; 25-40, 40-50, 50-60, greater than 70. They are shown on plates 9b to 11. They confirm the theory that two soil types exist. One is a medium plastic clay with a liquid limit test less than 40 and approximately 65% of its natural moisture content determination between 15% and 21%. The other is a highly plastic clay with a liquid limit greater than 50 and 60% of its natural moisture content determinations in the range 27% to 36%. The histogram for samples with liquid limits between 40 and 50 did not show any natural moisture content that was clearly defined as being more common than the others.

Further subdivision within the two soil groups that can be defined on the basis of liquid limit and natural moisture content has not been possible. That is to say for two samples of highly plastic clay, one having a liquid limit of 55 and the other 75, no rule has been found for saying which will have the higher moisture content. Probably both will have moisture contents between 27% and 36%.

5. Summary of Results -

a) Plasticity Chart -

- i. Most of the soils in the region are of the same geological origin.
- ii. According to the Casagrande system the soils in the area are clays of intermediate to high plasticity (CI to CH).

b) Histogram of Liquid Limit -

- i. On the basis of liquid limit, there are two types of cohesive soil in the region.
- ii. One is a clay of intermediate plasticity (liquid limit 30-40).

iii. The other is a clay of high plasticity having liquid limits commonly between 60 and 65 and occasionally as high as 95.

iv. The highly plastic clay occurs more frequently.

c) Liquid Limit Within Six Feet of the Surface -

The soils sampled from a depth of six feet or less had nearly the same frequency distribution of liquid limit as the soils from greater depths.

d) Natural Moisture Content -

i. On the basis of natural moisture content there are two types of soil in the region.

ii. They correspond to the two soil types indicated on the basis of liquid limit.

iii. The results of the two tests can be summarized as: -

Liquid Limit	Natural Moisture Content
Less than 40	15% to 21%
Greater than 50	27% to 36%

In each case over 60% of the natural moisture content tests were within the range given above.

VIII UNCONFINED COMPRESSIVE STRENGTH

The objective was to find if any relationship exists between unconfined compressive strength and natural moisture content. For this purpose plots of natural moisture content against the logarithm of the unconfined compressive strength were used. In the discussion which follows it should be understood that whenever a moisture content vs. unconfined compressive strength plot is mentioned, the moisture content is to be plotted to an arithmetic scale and the unconfined compressive strength to a logarithmic scale.

The following moisture content-unconfined compressive strength plots were made : -

i. A random sample of the test data;

ii. The test results from one site for which a large amount of data was available;

iii. The data from (ii) grouped according to the degree of saturation of the samples;

iv. Test results for soils from the same region on the plasticity chart;

v. A comparison of the best fit lines from (iv).

1. A Random Sample of the Test Data -

For the first trial a series of unconfined compressive strength tests and corresponding moisture contents were selected at random and the natural moisture content was plotted against the logarithm of the unconfined compressive strength. The objective of this plot was to attempt to confirm the results published by the United States Engineer Department (Ref. 20). No distinction was made between the various soil samples; all test results

were plotted on the same sheet.

The results of a number of points plotted in this manner are shown on plate 12. It indicates a general trend towards lower unconfined compressive strength at higher moisture content, but the scattering of the data is so large that it appears that other factors influence the results.

2. All Data From One Particular Site -

In an effort to find the factors causing the scatter of data, the unconfined compressive strength vs. moisture content plot was made for one site from which a large amount of data was available (plate 13). This helped to reduce the scatter of the plotted points, but not sufficiently to indicate that the main factor causing the scatter had been located. The small range of moisture content found at the site made it difficult to determine the best fit line for the plotted data.

3. The Data Used in (2) Grouped According to Degree of Saturation -

The results of the U.S.E.D. tests indicated that the degree of saturation has considerable effect on the unconfined compressive strength-moisture content plot. To investigate this result the data used in part 2 above was divided on the basis of degree of saturation. All test results for soils which had a degree of saturation in excess of 98% were plotted in one group; those from 95% to 98% saturation in a second group; and those below 95% saturation in a third group. The results of this process are shown on plate 14. This divisioning still does not reduce the scatter of the data appreciably. In addition, the small range in moisture content at this site makes the choice of a best fit line difficult.

4. Test Results Grouped According to the Position of the Soils on the Plasticity Chart -

In a further effort to reduce the scatter of the unconfined compressive strength-moisture content plots, separate plots were made for soils from the same zones on the plasticity chart. The zoning system used to divide the plasticity chart is shown on plate 7. This method of grouping reduced the scatter of the data considerably. On this basis the data was sufficiently consistent so that straight line relationships between moisture content and the logarithm of the unconfined compressive strength could be drawn for all the zones.

The results of the plots are shown on plates 15, 16, 17, 18 and 19. For each plot the best straight line relationship has been indicated. The two plots on each plate are both for soils within the same range of liquid limit. In each case part (a) is for soils from between the "A" and "B" lines and part (b) is for soils from above the "B" line.

Although grouping of test results on the basis of position on the plasticity chart does reduce scatter, some test results still fall far below the average. These points were investigated, keeping in mind the following factors which reduce the apparent unconfined strength.

i. The presence of sand lenses in the test specimen;
ii. Sample disturbance due to sampling;
iii. Faulty laboratory technique.

The following factor will tend to produce test results which indicate too high an unconfined compressive strength.

i. Slight drying of the specimen.

For the data used in the preparation of this report the unconfined compressive strength tests were generally run soon enough after sampling to

eliminate drying of the soil prior to testing. Therefore, it will be seen that all factors which influence the unconfined strength tend to decrease the apparent strength of the soil.

Any of the test results which showed low strength and which could be shown to probably be in error were marked on the plots and the reason for disregarding them noted. Since the effect of any error which may have been made but not noticed is to reduce the apparent unconfined compressive strength the best fit lines shown on the plots represent conservative values.

5. Comparison of Best Fit Lines From (4) -

The best fit lines for the various zones are compared on plate 20. Plate 20a shows the results for samples from between the "A" and "B" lines and plate 20b contains the results for samples above the "B" line.

From the results on this plate it can be seen that there is a general tendency for the unconfined compressive strength to decrease with decreasin liquid limit for soils at the same moisture content. This effect is most noticeable between the liquid limit ranges 25-40, 40-50 and 50-60. The results reconfirm the existence of two soil types originally identified from their liquid limits; in general soils with liquid limits less than 40 have one relationship between compressive strength and moisture content and those above 50 have another. Referring to plate 18b for the soils having liquid limits between 40 and 50 it will be seen the plotted points fall into two groups, indicating that some of the soils in this group follow the same moisture content relationship as the soils with liquid limits below 40 and the rest fit into the group having liquid limits above 50.

Comparing parts (a) and (b) of plate 20, it will be seen that this figure confirms Casagrande's results that soils with the same liquid limit and with plasticity index increasing, the unconfined compressive strength, at the same moisture content, increases.

The U.S.E.D. research also indicates that the straight line relationship between the unconfined compressive strength and moisture content is parallel to the virgin branch of the consolidation curve. For Edmonton soils the slope of the consolidation curve at the highest consolidation pressures commonly used is much flatter than the slope of the unconfined compressive strength-moisture content plot. This may be due to the preconsolidation of the Edmonton clay. The effect of preconsolidation can be removal by remolding the soil. On plate 20 there is shown a line parallel to the consolidation curve for a sample of Edmonton soil (W_L 65 PI 38) remolded and consolidation from a moisture content equal to the liquid limit by Curtis & Dickson (Ref. 7). This curve is very nearly parallel to the unconfined strength-moisture content plot. It should be noted that the consolidation curve shown here has been drawn parallel to the true virgin branch of the consolidation curve, for purposes of comparison. The actual consolidation curve lies approximately one cycle of log paper to the right of the line shown.

6. Summary of Results -

a) For all soils in the area -

i. Plate 12 indicates that a relationship exists between moisture content and unconfined compressive strength.

ii. The large scatter of the plotted points suggests that some other factor also influences the results.

b) Soils from one particular site -

- i. This procedure does not result in any improvement over case a.
- ii. The small range of moisture content at the site makes the selection of a best fit line difficult.

c) Grouping the data used in b) on the basis of degree of saturation - This procedure does not reduce the scatter of the plotted points appreciably.

d) Grouping soils from the same zone on the plasticity chart -

- i. This procedure reduces the scatter of the data considerably.
- ii. It was possible to determine a straight line relationship for each group of plotted points.

e) Comparison of the best fit lines determined in d) -

From the results shown on plate 20, the following conclusions can be drawn: -

- i. There is a tendency for the unconfined compressive strength to decrease with decreasing liquid limits for samples at the same moisture content.
- ii. This tendency is most marked for liquid limits in the intervals from 25-40, 40-50 and 50-60.
- iii. Soils with liquid limits greater than 50 have one general relationship between compressive strength and moisture content. Those with a liquid limit below 40 have another relationship.
- iv. Soils with liquid limits between 40 and 50 fit into either the group below 40 or the group above 50.

v. Comparing soils of the same liquid limit with plasticity index increasing, the unconfined strength increases for samples of equal moisture content.

vi. The only consolidation test available in which an Edmonton clay (W_L 65 PI 38) was remolded and consolidated from its liquid limit had a straight line portion approximately parallel to the moisture content-unconfined compressive strength curve.

IX CONSOLIDATION CHARACTERISTICS

The objective of this series of analyses was to see if any general conclusions could be drawn concerning the results of consolidation tests on Edmonton soils. The following items were investigated: -

i. Preconsolidation load;

ii. Compressive index.

1. Preconsolidation Load -

The majority of the soils in the Edmonton area have been preconsolidated. That is, at some time they have been subjected to larger loads than the ones which exist at the present time. The high stresses were produced in either of two ways, by the weight of a glacier loading material laid down during a previous glaciation, or by high capillary pressures exerted on the soil during drying.

Geologic evidence shows that three separate glaciations occurred in the area. Each glacier was in place long enough to consolidate the soil laid down during the previous period. During the retreat of the last glacier, the Edmonton area was covered by a shallow lake. In many places the present surface soils were laid down on the bottom of this lake. These soils have been subjected to preconsolidation pressures exerted during drying of the soil. Evidence that desiccation of the soil has occurred is the nugget structure commonly found near the surface. The combined effect of glaciation and desiccation has been to preconsolidate most of the soils in the area.

It is known that for foundations on preconsolidated clay, settlement will generally not be a problem if the foundation pressure is less than the preconsolidation load. The actual value of the preconsolidation

load is therefore of considerable significance in foundation engineering.

If it were possible to determine the thickness of the glacier which covered the area, an estimate of the preconsolidation load could be made. Unfortunately, in the Edmonton area, the glaciers melted without leaving any visible trace of their thickness. However, in southern Alberta one range of hills, known as the Cypress Hills, was never overtopped by a glacier. From the position of the glacial deposits on these hills, it can be ascertained that the glacier in the Cypress Hills area was about 1500 feet thick. If 1500 feet is taken as the probable minimum thickness of glacier that could be expected in Edmonton, then the minimum preconsolidation pressure due to the weight of the glacier would be 42 tons per square foot. This is a very high preconsolidation load.

After estimating the preconsolidation load from geologic evidence, the results of consolidation curves for Edmonton soils were investigated to see if they confirmed or contradicted the estimated minimum preconsolidation load of 42 tons per square foot.

An attempt was made to see if the data available indicated that the preconsolidation load had been reached in the laboratory test with loads of 19.2 tons per square foot. This method used the known result that below the preconsolidation load the slope of the void ratio vs. the logarithm of the consolidation pressure curve increases with increasing pressure and becomes constant at some load in the vicinity of the preconsolidation load. Accordingly the consolidation curves available were investigated to see if the slope of the curve was still increasing at the maximum load used in the tests. To obtain an average result consolidation test data was selected so that identical consolidation loads had been used for all the tests. The average slope of the chord between adjacent consolidation loads was used as a

measure of the slope of the curve between each of the consolidation pressures. The results shown indicate that the slope of the consolidation curve is still increasing in the pressure increment 9.60 to 19.19 tons per square foot. This indicates that the preconsolidation load has not been reached.

Table II

Apparent Compressive Index at Various Loads

Results of 12 Tests

P_1 tons per sq. ft.	.16	.32	.64	1.28	2.40	4.80	9.60
P_2 " " " "	.32	.64	1.28	2.40	4.80	9.60	19.19
Apparent compressive index	.042	.071	.100	.143	.183	.248	.296
Change in compressive index		.029	.029	.043	.040	.065	.048

The reasoning used above will be in error if the increased slope is not due to consolidation but is caused by soil from the test specimen squeezing out past the loading head at the high loads. No remarks concerning soil squeezing past the loading head were made on the data sheets used and there were no sharp breaks in the consolidation curve to indicate that escape of soil from the consolidation ring had occurred. Therefore it is felt that the result in Table II represents a true property of the soil. On this basis the preconsolidation load for the soil tested is greater than 19.2 tons per square foot.

Another method of determining the preconsolidation load has been suggested by Casagrande (Ref. 2). It involves a geometrical construction on the pressure void ratio curve obtained in a consolidation test. To obtain the preconsolidation load using this construction, it is necessary

to know the slope of the virgin branch of the consolidation curve. None of the test results available was carried to a consolidation load of more than 19 tons per square foot which was felt to be below the preconsolidation load so Casagrande's construction could not be used.

2. Compressive Index -

From the results in Table II it appears as if the slope of the e-log p curve is still increasing at the maximum load available for most tests and therefore the slope of the e-log p curve below 19 tons per square foot is not the true compressive index of the soil.

Skempton (Ref. 17), suggests that a relationship between the compressive index and liquid limit exists. On the basis of the results of Skempton and others, Terzaghi and Peck (Ref. 19), advocate the use of the relationship $C_c = K\ (W_L - 10)$ for predicting the compressive index of a soil from its liquid limit. They suggest using K equal to .009 for undisturbed clays and .007 for remolded clays. This relationship was investigated for Edmonton soils by means of the plot shown on plate 21. On this point it will be noted that except for four undisturbed samples with liquid limits between 40 and 50 and two remolded samples the measured value of the compressive index are smaller than would be predicted by the relationship above. This result supports the theory that the soils in the Edmonton area are so highly preconsolidated that the virgin branch of the preconsolidation curve is not reached at a consolidation pressure of 19.2 tons per square foot.

In an effort to compute the true slope of the virgin branch of the consolidation curve the method suggested by Peck, Hanson and Thornburn

(Ref. 12) was used. Referring to plate 22 the method recommended is: -

i. Plot the undisturbed and remolded consolidation curves for the soil; K_u and K_r.

ii. Extrapolate the curves to meet at a point (f) on the zero void ratio axis;

iii. Draw a vertical line, through the known preconsolidation pressure p_o', and a horizontal through e_o, the natural void ratio of the undisturbed sample, to meet at point a';

iv. The point b is found by extending the virgin branch of the consolidation curve back to meet the horizontal line through e_o;

v. p_o is the effective overburden pressure;

vi. abf is the field consolidation curve.

Because no undisturbed test result was available for which the virgin branch of the consolidation curve was known, point f could not be determined by the procedure used above. Instead, f was located using the results of consolidation tests on four separate remolded samples. When extrapolated the four curves all met at a point on the zero void ratio axis as shown in plate 23. Since the soils had liquid limits of 54, 65, 65 and 73, it was considered that the location established for point f was valid for all soils from the Edmonton area with liquid limit greater than 50.

Next, the consolidation curves available were plotted on a sheet so that point f could be shown. They were grouped on the same basis as the moisture content-compressive strength data. All the consolidation curves for soils from the same zone on the plasticity chart were grouped together. Using point f determined from the four remolded curves, an estimated preconsolidation load of 42 tons per square foot and an average

initial void ratio for each set of curves; the field consolidation curve was determined. (Plates 24, 25 and 26). Because point f had not been established for soils with liquid limits less than 50, it was not possible to draw the field consolidation curves on plates 27 and 28.

The procedure just described involves some broad assumptions, which may not be justified. For this reason, the results are not presented as established facts but rather as indications of the order of compressive index which the Edmonton soils might exhibit at loads above the preconsolidation load. The only way in which these results can be verified is by carefully selecting undisturbed samples of soils from the Edmonton area and performing consolidation tests on them, in which the consolidation load is carried to 200 tons per square foot. Special consolidation equipment would have to be designed for this type of test.

Table III shows a comparison of the compressive index obtained in the manner described above; the slope of the moisture content-unconfined compressive strength curve; which research has shown to be parallel to the virgin branch of the consolidation curve; and the slope of the consolidation curve for a sample of Edmonton clay remolded and consolidated from a moisture content equal to its liquid limit. They agree quite well, considering the assumptions made in their determination.

TABLE III

Comparison of Computed Values of the Compressive Index

Liquid Limit	(1)	(2)	(3)
50 - 60	0.56	0.66	
60 - 70	0.56	0.64	0.58
greater than 70	0.62	0.59	

(1) Computed compressive index.

(2) Slope of the moisture content-unconfined compressive strength plot.

(3) Compressive index for a sample remolded and consolidated from a moisture content equal to the liquid limit ($W_L = 65$).

3. Results -

The results of the investigation are tentative only. They indicate the desirability of a consolidation test on Edmonton clay for which very high consolidation pressures are used. The object of the test would be to verify the following results obtained by analysis of the existing data : -

i. In laboratory consolidation tests conducted to a consolidation load of 19.2 tons per square foot, the virgin branch of the consolidation curve was not reached;

ii. From geological considerations it is estimated that the preconsolidation load for the highly preconsolidated soils in the Edmonton area is at least 42 tons per square foot;

iii. The true slope of the virgin branch of the consolidation curve for the soils found in the area is approximately 0.60.

X SWELLING PRESSURE

In the Edmonton area, many of the foundation problems are caused by swelling of the soils. The swelling takes place due to the ability of the clay materials in the soil to take water into their crystal structure in the presence of free water.

A test has been devised to give an indication of the swelling pressure which a soil may exert in the presence of free moisture. In this test an undisturbed sample of the soil is placed in the consolidation apparatus under a very light load. The soil is submerged and allowed to swell. After swelling is complete, the load required to return the sample to its original volume is recorded as the swelling pressure. (A more detailed procedure for the consolidation test run as a swelling test will be found in Appendix B).

Some doubt has been raised about the validity of the numerical value of the swelling pressure measured in this way, but the results are useful qualitatively if not quantitatively.

In this investigation the object was to find a simple way to decide which soils will exert a high swelling pressure. The following correlations were attempted: -

i. Swelling pressure vs. liquid limit;
ii. Swelling pressure vs. natural moisture content;
iii. Swelling pressure vs. liquidity index;
iv. Volume change during swelling $\frac{(\Delta e)}{(1+e)}$ vs. liquidity index.

1. Swelling Pressure vs. Liquid Limit -

The results of the plot are shown on plate 29. This plate indicates that soils with high liquid limits generally exert the highest swelling

pressures, but some of the soils with very high liquid limits exert little or no swelling pressure. The scatter of the plotted points on this figure is too great to make this correlation useful for predicting swelling pressures.

2. Swelling Pressure vs. Natural Moisture Content -

This plot shown on plate 30 gives somewhat better correlation than the one using liquid limit but there is still a large scattering of the plotted points. The soils with low moisture contents generally exert the highest swelling pressures.

3. Swelling Pressure vs. Liquidity Index -

The results shown on plates 29 and 30 suggest that there is some correlation between natural moisture content, Atterberg limits and swelling pressure. The liquidity index was selected as a property which might be related to swelling pressure. The results of a plot of swelling pressure against liquidity index are shown in plate 31. This plot indicates that a relationship exists although the scatter of the data is so large that no exact relationship can be drawn.

On this plot the liquid limit range in which each of the samples tested lies is indicated by the shape of the plotted point and the points are divided into one group that lies above the "B" line and another group that plots above the "A" line and below the "B" line on the plasticity chart.

The following deductions can be made: -

i. Soils with liquid limits less than 40 exert little or no swelling pressure;

ii. The other soils exert increasing swelling pressure with decreasing plasticity index;

iii. The closed points indicating soils from between the "A" and "B" lines on the plasticity chart fall generally within the band marked (1);

iv. The open points indicating soils from above the "B" line generally fall within band (2);

v. Band (2) is above band (1), indicating that at the same liquidity index, the soils with the higher plasticity index will exert the higher swelling pressure;

vi. During the swelling test the points marked with a stroke (') were allowed to swell at a smaller load than the other points. They indicate that the swelling pressure test is sensitive to the load under which swelling takes place. A higher swelling pressure will be measured if the soil is allowed to swell at a smaller load.

The deductions listed above require further investigation. In particular, part vi requires more data to confirm or deny it.

A large amount of the scatter of the plotted points may be due to errors in the value of the liquidity index. Casagrande and Fadum (Ref. 4), has shown the care necessary to get consistent values of the liquidity index. Because of the large possible errors in the liquidity index it was considered that a more detailed investigation of the relationship between swelling pressure and liquidity index could not be made with the data available. If future investigation of this relationship is carried out, special effort should be made to get an accurate value for the liquidity index. This will mean carefully selecting moisture content samples and Atterberg limit samples from the same soil as is used for the swelling pressure test.

4. Volume Change During Swelling vs. Liquidity Index -

At the same time as the swelling pressure liquidity index relationship was investigated, a plot was made of volume change during swelling $\frac{(\Delta e)}{(1+e)}$ against liquidity index (plate 32). The purpose of this plot was to see if it was the volume change during swelling rather than the swelling pressure that was a function of the liquidity index of the soil. The results show no significant correlation between these properties.

5. Results -

a) Soils with liquid limits less than ~~5~~40 exert little or no swelling pressure.

b) Swelling pressure increases with decreasing liquidity index.

c) Comparing soils at the same liquidity index, the one with the higher plasticity index will exert the higher swelling pressure.

d) The results of the swelling pressure test depend on the pressure applied to the soil during swelling.

XI MODULUS OF DEFORMATION

The modulus of deformation of a soil is defined as the ratio of the applied stress to the corresponding decrease in length per unit length, of the specimen. The term modulus of deformation rather than modulus of elasticity is used in order to avoid implying that the behaviour of the soil is elastic.

The value of the modulus of deformation determined in a laboratory test depends on the test conditions. In particular, the lateral restraint to which the sample is subjected will have an effect on the modulus. The two limiting cases are encountered in the unconfined compression test where the specimen has no lateral restraint and in the consolidation test where all lateral movement of the specimen is prevented. The conditions encountered in nature will lie somewhere between the fully restrained consolidation test and the unconfined compression test.

An analysis of the modulus of deformation in confined compression was made for Edmonton clay soils. The modulus was calculated from the results of the consolidation test. The change in void ratio during consolidation can be converted to unit deformation by means of the relationship

$$\text{unit deformation} = \frac{\Delta l}{l} = \frac{\Delta e}{1 + e_o}$$

The modulus of deformation in confined compression between any two applied pressures can then be calculated from the results of the consolidation test without actually plotting the stress-deformation curve because $M = \dfrac{\Delta P}{\dfrac{(\Delta e)}{(1+e_o)}}$

where M is the modulus of deformation

P is the pressure increment

The results of the calculation of the modulus of deformation for Edmonton clays for three pressure increments are shown in Table IV.

Table IV

Modulus of Deformation in Confined Compression

Pressure Increment (tons per sq ft)	Modulus of Deformation (tons per sq ft)		
	Max.	Min.	Avg. (40 tests)
.08 to 1.0	460	8.9	65
.08 to 3.0	264	17.5	66
.08 to 8.0	165	36.0	91

Swelling of the soil in the presence of free water is the cause of the maximum values of the modulus of deformation being so high compared to the average value for the first two cases above. The loading head of the consolidation apparatus not being firmly seated on the specimen at the start of the test was probably the cause of the lowest values. The average values indicate that unlike most load deformation curves, the curves for clay in confined compression are concave upwards.

Settlements under very heavy building loads on the Edmonton soils often occur very quickly and then stop. It is thought that these settlements are due to elastic compression of the soil. To investigate the magnitude of this rapid compression relative to the long-term consolidation of the soil, the laboratory time-settlement readings were analyzed. The settlement which took place with 6 seconds after the load increment was applied was taken as the initial compression. For each load increment used during the test the initial compression was expressed as a percent of the total settlement under the load. In order to obtain representative values the results of several tests were averaged. For purpose of comparison the same procedure was followed for a lean clay (W_L 40, PI 18) and a slightly plastic silt W_L 22, PI 6) from Kitimat. The results of these calculations are shown as load-deformation curves on plate 33 and are tabulated in plate 34.

The results shown in these plates indicate that the load deformation properties of the two clays and the silt are quite different. Comparing the preconsolidated Edmonton clay with the presumably unpreconsolidated Kitimat clay, it can be seen that the deformation of the Kitimat clay under load is only slightly greater than for the Edmonton clay but a much larger portion of the deformation of the Edmonton clay takes place within six seconds of loading than occurs for the Kitimat clay. This indicates that a large part of the deformation of the preconsolidated Edmonton clay is due to elastic deformation. It should also be noticed that the percentage of the deformation which takes place within six seconds decreases with increasing load for the highly swelling Edmonton soil but remains practically constant after the first load increment for the Kitimat soils. One possible explanation of this behaviour is that the swelling tendency of the Edmonton clay is strong enough to resist consolidation at light loads but is unable to prevent consolidation under heavier loads. The deformation that occurs at small loads is therefore practically all initial compression whereas the deformation at high loads is partly initial compression and partly consolidation. The non-swelling Kitimat clay on the other hand does not show any strong tendency towards either more or less initial compression with increasing load except after the first load increment for which the large initial compression is probably due to the loading head becoming firmly seated on the sample.

The silt is less compressible than either ~~the silt~~ of the clays and examination of the time-settlement curves indicates that part of the primary consolidation ~~had occurred~~ occurs within six seconds of the load application. This accounts for the relatively high proportion of the deformation which occurs within six seconds of the application of load to the sample.

XII BLOW COUNT ON THE SAMPLING TUBE

During the field sampling program, a record is kept of the number of blows of a 30 pound hammer dropped 30 inches that are required to drive a 3 inch diameter Shelby tube various distances into the soil. The results of this test are used as a check on the unconfined compressive strength and natural moisture content data obtained in the laboratory. In an effort to show a correlation between the blow count and the unconfined strength or the natural moisture content test results, various plots were made:

i. Blow count vs. unconfined compressive strength for all samples available.

ii. Blow count vs. unconfined compressive strength for soils with liquid limits less than 50.

iii. Blow count vs. unconfined compressive strength for soils with a liquid limit greater than 50.

iv. The logarithm of the blow count vs. moisture content.

In each case the blow count used was the number of blows required to drive the sample tube a distance of 0.8 feet into the soil. This particular penetration distance was used because it was the one for which the most data ~~was~~ were available.

1. Blow Count vs. Unconfined Compressive Strength -

The plot is shown on plate 35. It shows a good correlation between blow count and unconfined compressive strength for strengths less than 3.0 tons per sq ft. Accordingly the following approximate relationship is suggested when the blow count is less than 70,

$$q_u = .04\ B$$

where q_u is the unconfined compressive strength and B is the number of blows

of a 30 pound hammer dropped 30 inches, required to drive a 3 inch diameter Shelby tube 0.8 feet into the soil.

This relationship is generally on the conservative side although the maximum error on the unsafe side is 100%. This error occurs for soils containing small lenses of sand or gravel which tend to decrease the apparent compressive strength of small laboratory samples to a much greater extent than they actually affect the shear strength of the large mass of soil in the field.

It is not suggested that the above relationship will make unconfined compressive strength tests unnecessary but rather that it can be used for a preliminary estimate of the bearing capacity of the soil which can later be supplemented by unconfined compressive strength tests.

2. Blow Count vs. Unconfined Compressive Strength for Soils with a Liquid Limit Less than 50 -

In an effort to reduce the scatter associated with the previous plot, separate sheets were used to show the results forsoils having liquid limits above and below 50. The plot for liquid limits less than 50 (plate 36), does show some reduction of the scatter in the data. This may be mainly because there are very few strength test results greater than 3 tons per square foot in this group.

3. Blow Count vs. Unconfined Compressive Strength for Soils with Liquid Limit Greater than 50 -

The results of this plot are shown in plate 37. The scatter of the data on this plate is worse than on either of the two previous plots. Use of the relationship suggested above is still recommended for a preliminary estimate of the unconfined compressive strength because it generally

provides a conservative answer.

4. Moisture Content vs. Logarithm of the Blow Count -

This is the same type of plot as is used for moisture content and unconfined compressive strength. The results are shown on plate 38. On this plot there is some trend towards lower blow counts at higher moisture contents. Unfortunately too many of the blow count results fall within the range 25-60 at all moisture contents so that no definite relationship can be obtained.

5. Results -

a) The relationship $q_u = .04$ B can be used as a rough estimate of the results of most laboratory unconfined compressive strength tests for blow counts of less than 70.

b) For soils with a high unconfined compressive strength, i.e., greater than 3 tons per square foot, the results of a blow count test have not been correlated with the unconfined compressive strength.

XIII CONCLUSIONS

The following conclusions about the properties of the soils in the Edmonton area have been reached.

1. On the basis of liquid limit and natural moisture content, two soil types can be distinguished. According to the Casagrande system of classification one is a clay of high plasticity (CH) and the other is a clay of intermediate plasticity (CI).

2. No general statement can be made concerning which type of soil will be encountered at any particular location.

3. A straight line relationship between moisture content and the logarithm of the unconfined compressive strength of the soil exists. The relationship is different for soils from the different locations on the plasticity chart.

4. Most of the soils in the area have been preconsolidated either by the weight of glaciers or due to desiccation.

5. In the consolidation test on Edmonton soils the virgin branch of the consolidation curve is not reached when undisturbed samples are tested to a maximum pressure of 19.2 tons per square foot.

6. Soils that were consolidated due to the weight of glaciers have been preconsolidated under a load estimated from geological evidence to be at least 42 tons per square foot.

7. Swelling pressure increases with decreasing liquidity index.

8. For soils at the same liquidity index the swelling pressure increases with plasticity index.

9. By counting the number of blows of a standard hammer falling a standard distance required to drive a standard sampling spoon to 0.8 feet penetration, an indication of the unconfined compressive strength of the soil can be obtained from the approximate relationship $q_u = .04B$ when the unconfined compressive strength is less than 3 tons per square foot.

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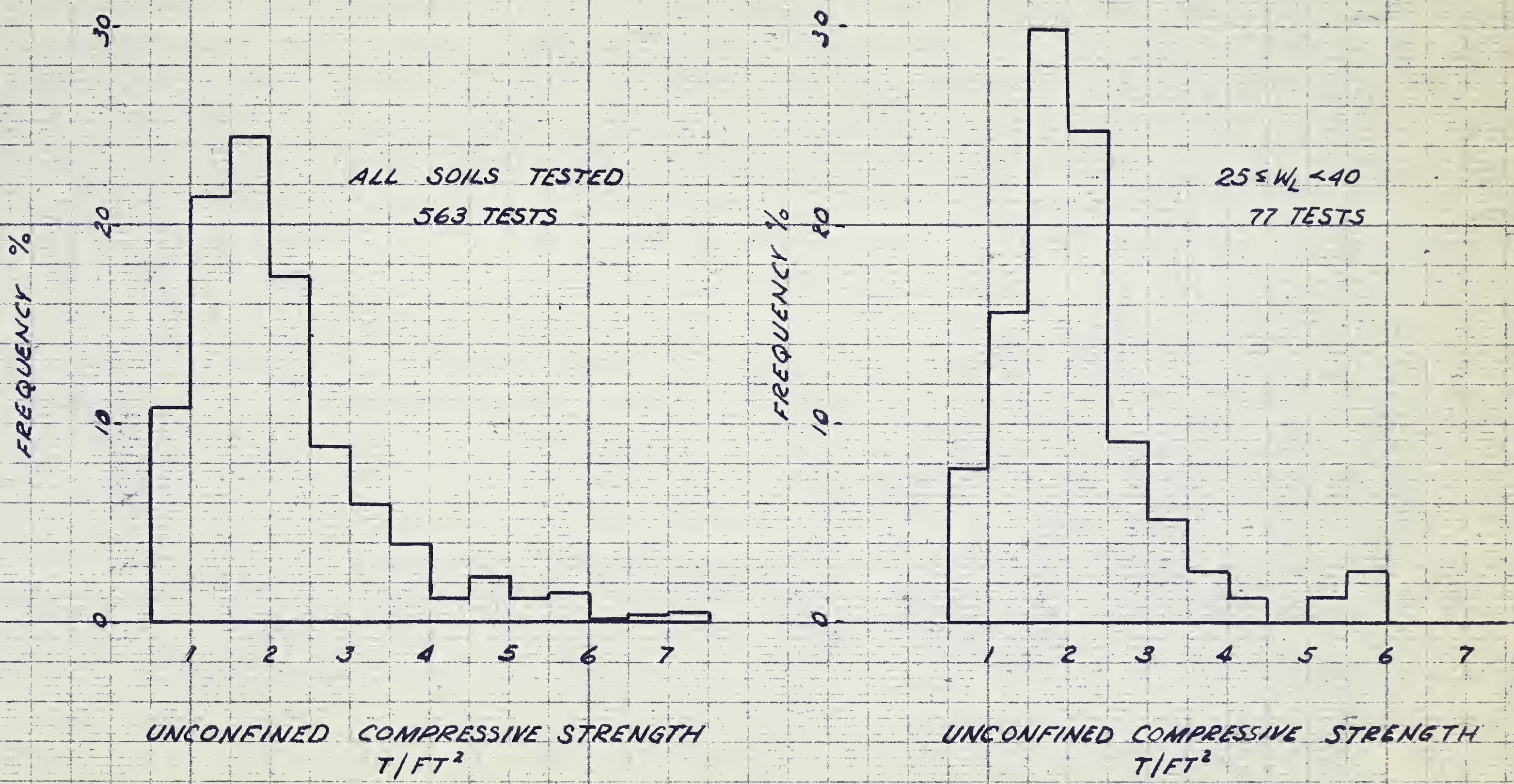
Table I

TABLE OF FORMATIONS

System	Series	Formation	Description
Quaternary	Recent	Postglacial	Soils and lake clays.
		Silt Till	Light buff coloured silty till, locally without pebbles.
	Pleistocene	Interglacial	Cross-bedded sands with some dark carbonaceous bands.
		Brown Till	Brown columnar till with pebbles dominately silt-clay. Keewatin types -- granite & gneiss.
		Interglacial	Medium-sized brown sands grading upward into gravels.
		Grey Till	Grey-coloured plastic clay with coal fragments and pebbles. Keewatin types -- granite and gneiss.
		Preglacial "Saskatchewan" sands & gravels	Gravels and sands. Cordilleran arkose, quartzites and chert pebbles.
Cretaceous	Upper	Edmonton	Sandstone, bentonitic shales, coal seams.
	After G.A. Collins and A.G. Swan, (1955)		

PLATE # 1



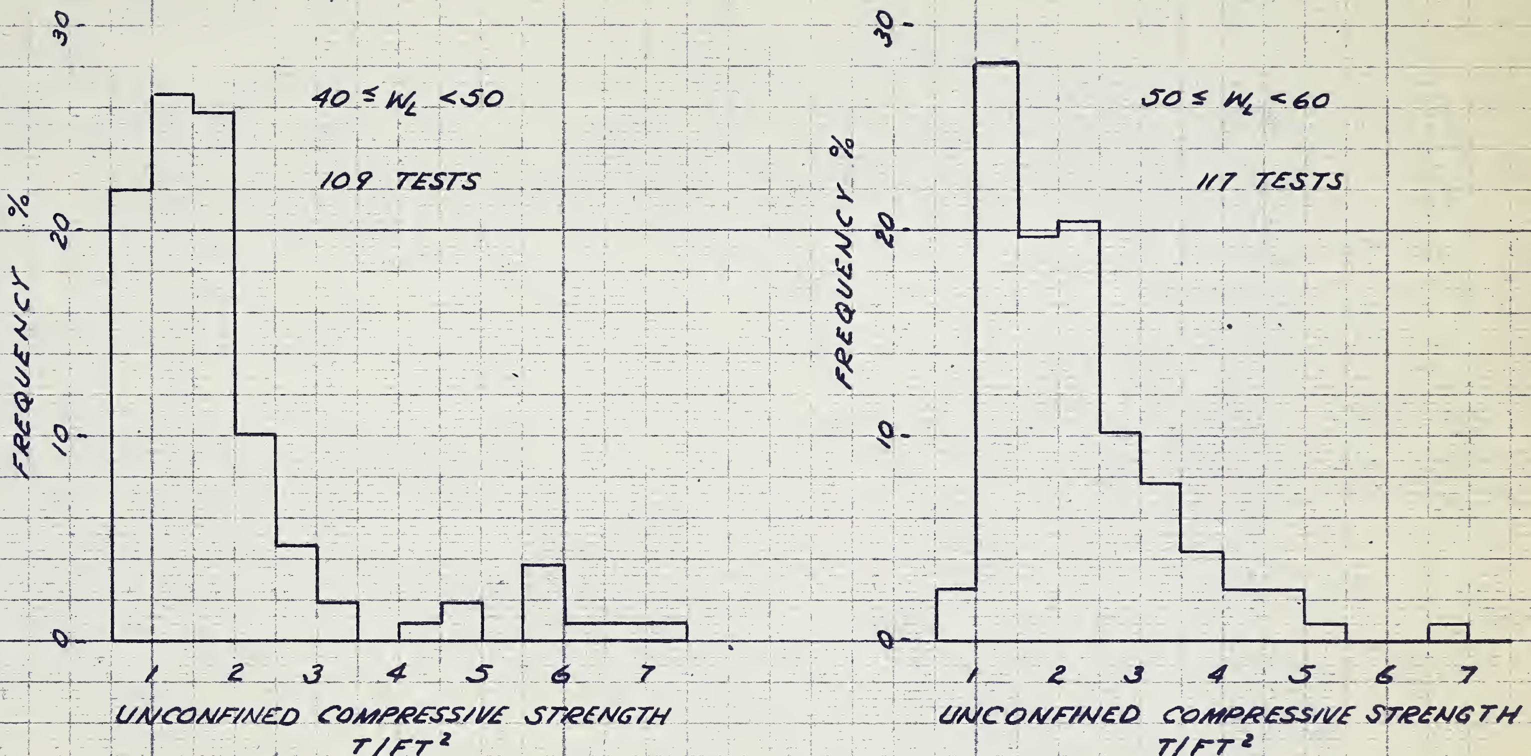
HISTOGRAM OF UNCONFINED COMPRESSIVE STRENGTH

(a)

(b)

PLATE #2

PLATE #2

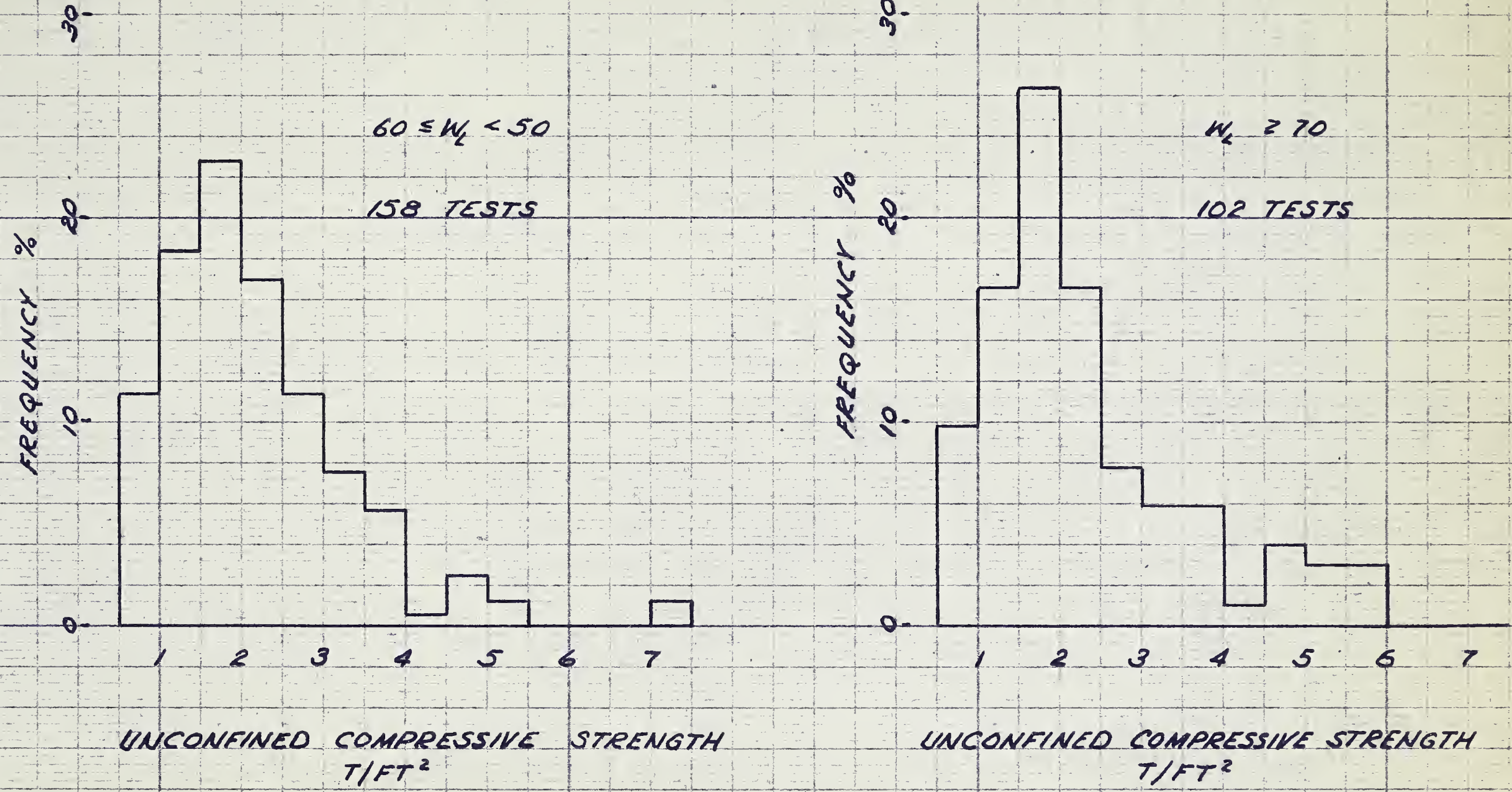


HISTOGRAM OF UNCONFINED COMPRESSIVE STRENGTH.

(a) (b)

PLATE # 3

PLATE # 3



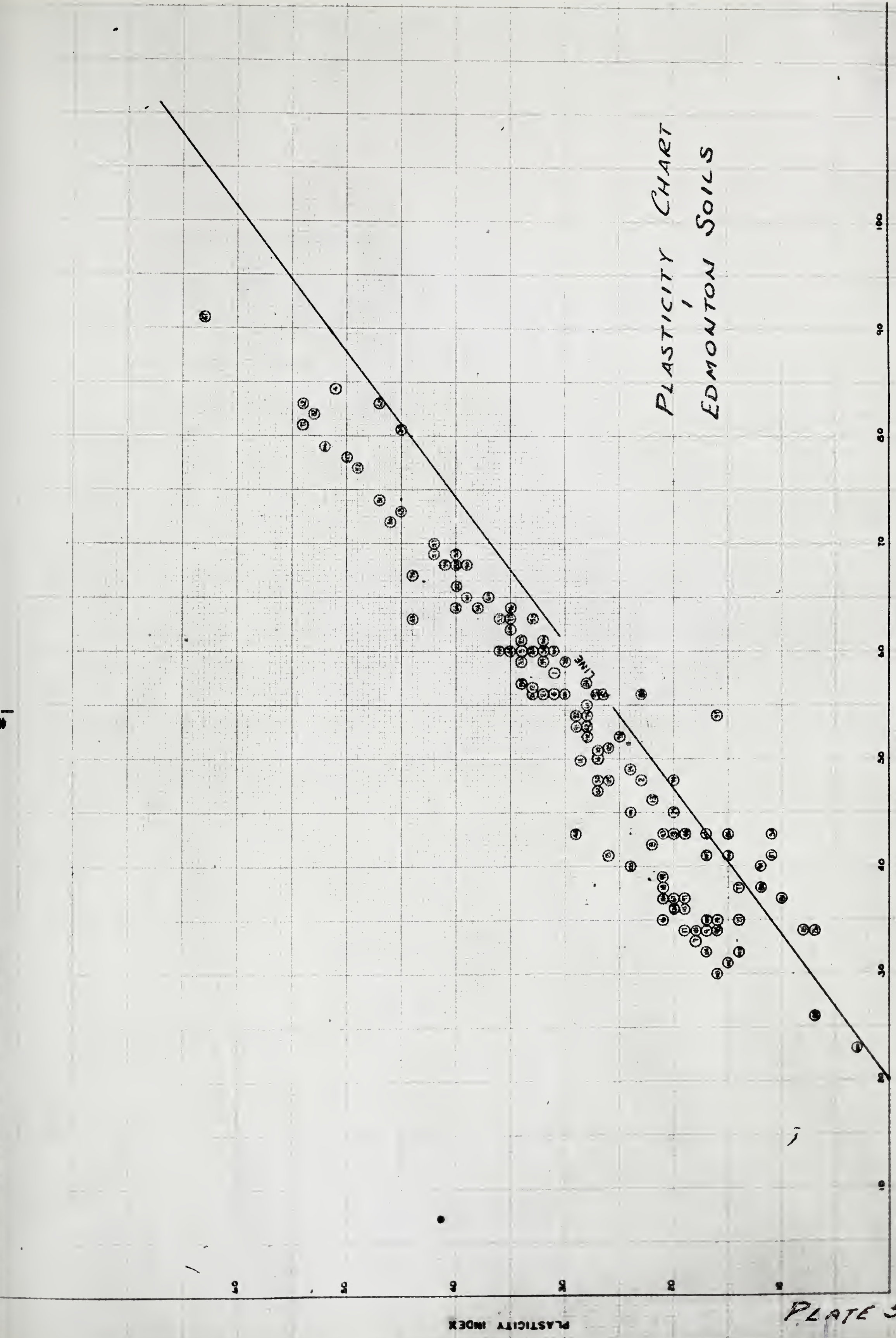
HISTOGRAM OF UNCONFINED COMPRESSIVE STRENGTH

(a) (b)

PLATE * 4

PLATE * 4

PLATE 5



#1

PLATE 5

PLASTICITY CHART
EDMONTON SOILS
#2

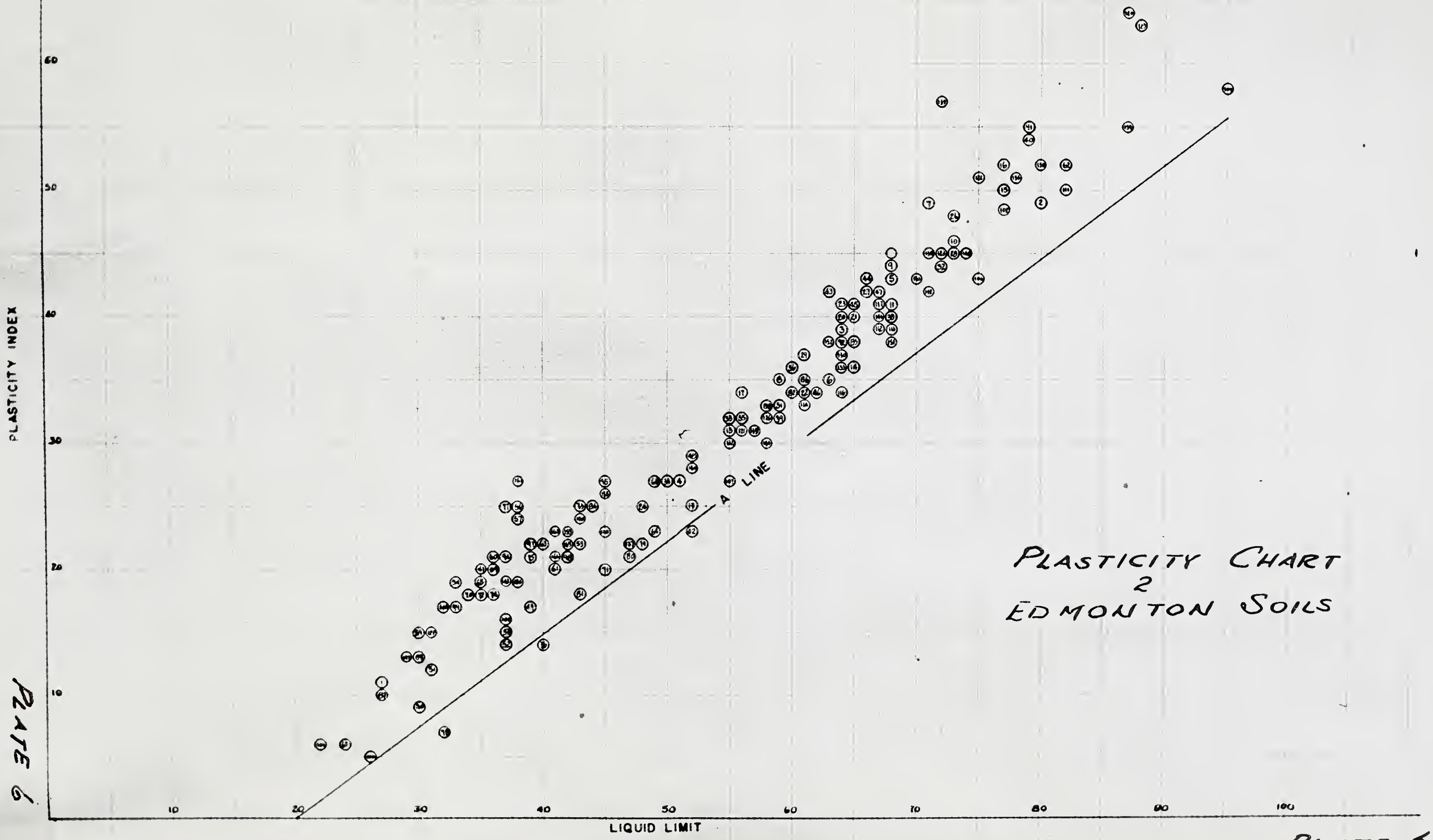
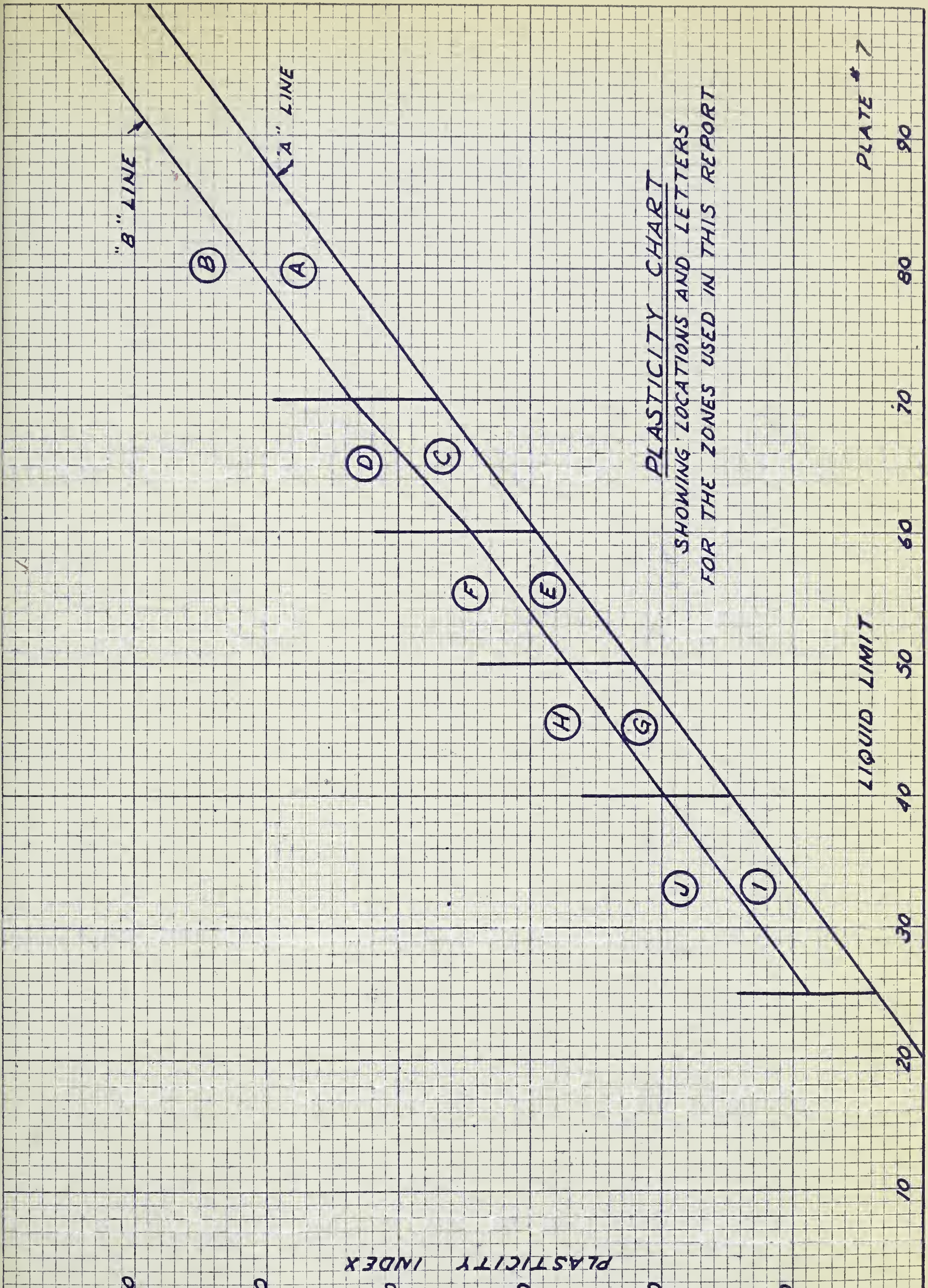


PLATE 6

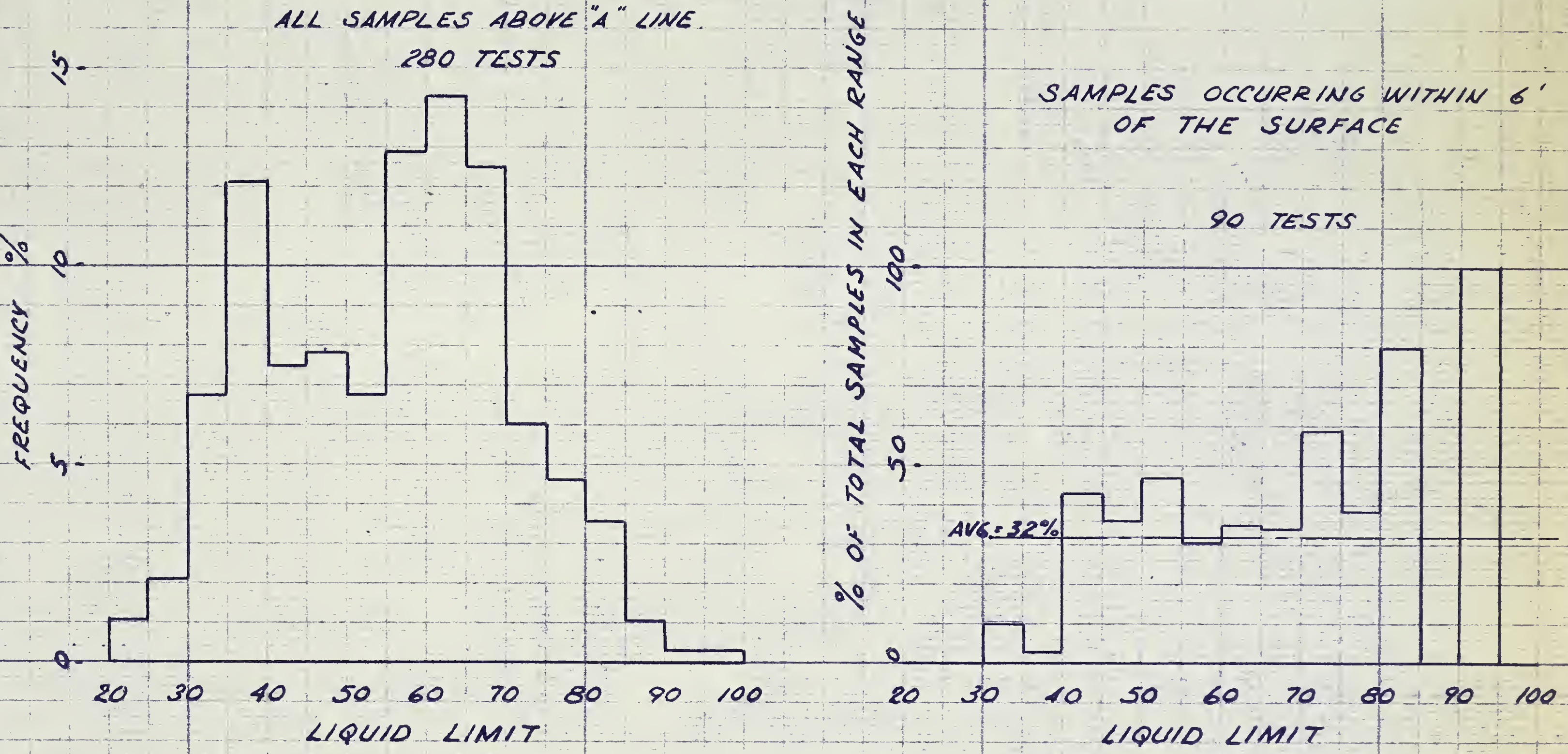
PLATE 6



PLASTICITY CHART

SHOWING LOCATIONS AND LETTERS FOR THE ZONES USED IN THIS REPORT

PLATE # 7



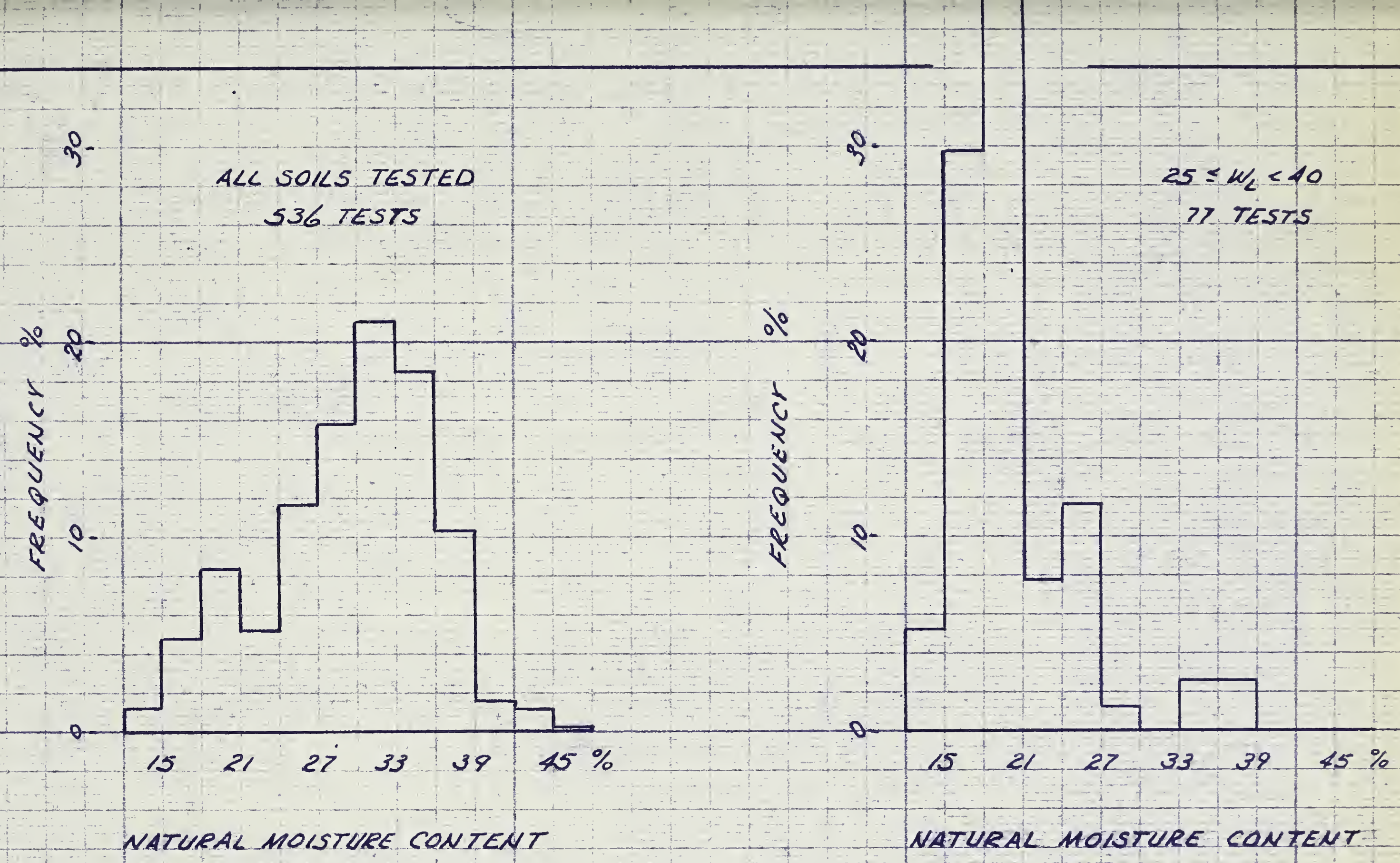
HISTOGRAM OF LIQUID LIMIT

(a)

(b)

PLATE #8

PLATE #8

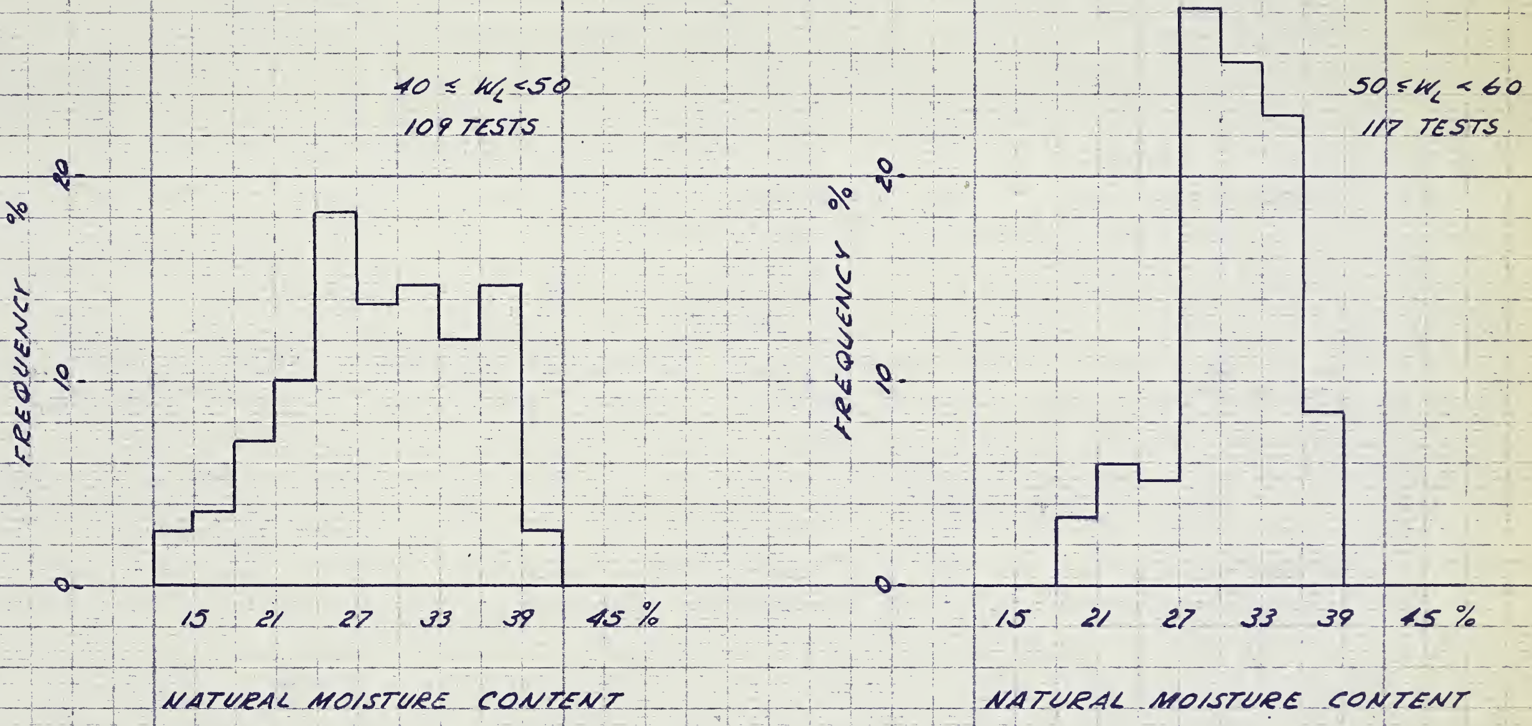


(a) (b)

HISTOGRAM OF NATURAL MOISTURE CONTENT

PLATE # 9

PLATE # 9



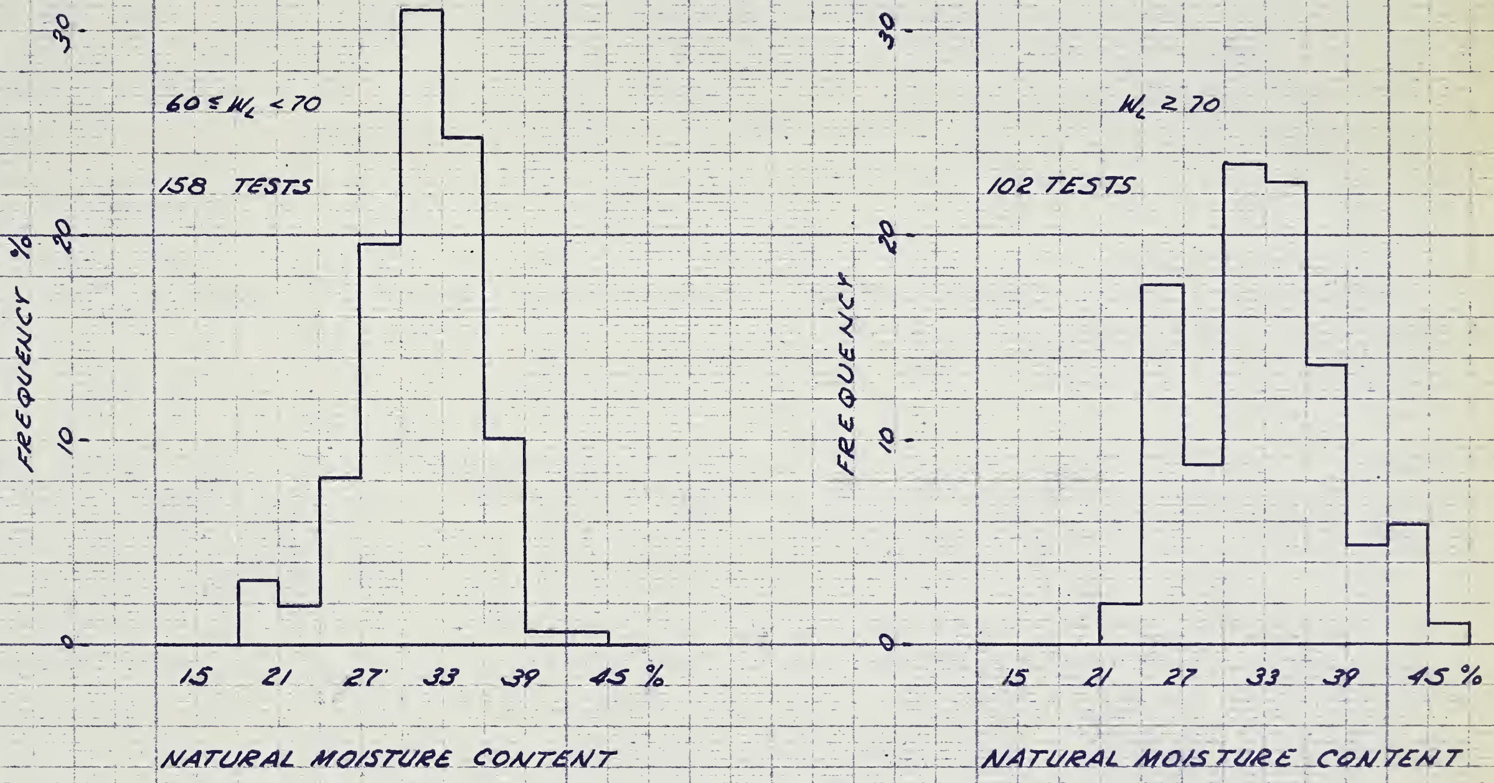
HISTOGRAM OF NATURAL MOISTURE CONTENT

(a)

(b)

PLATE # 10

PLATE # 10

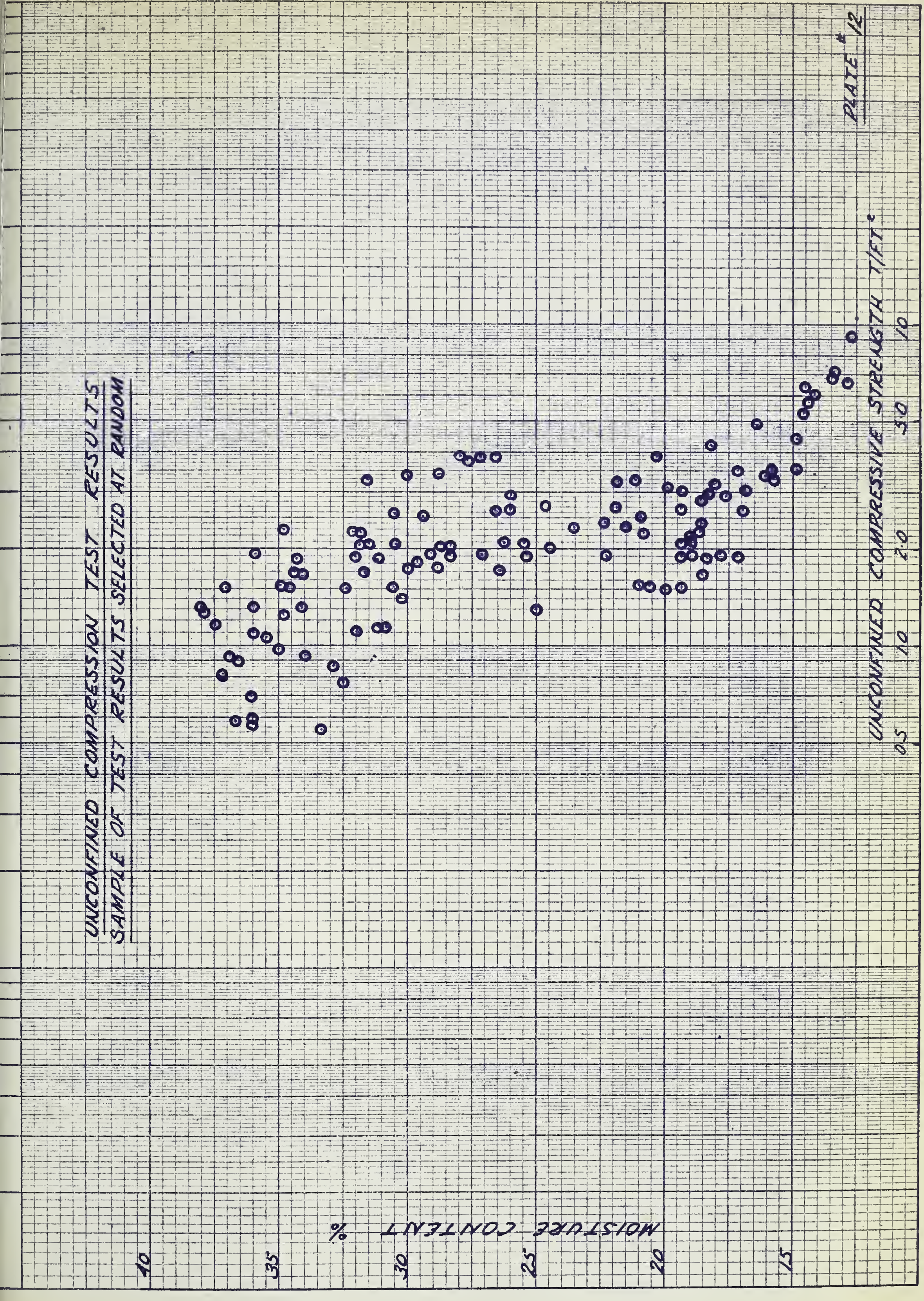


HISTOGRAM OF NATURAL MOISTURE CONTENT

(a) (b)

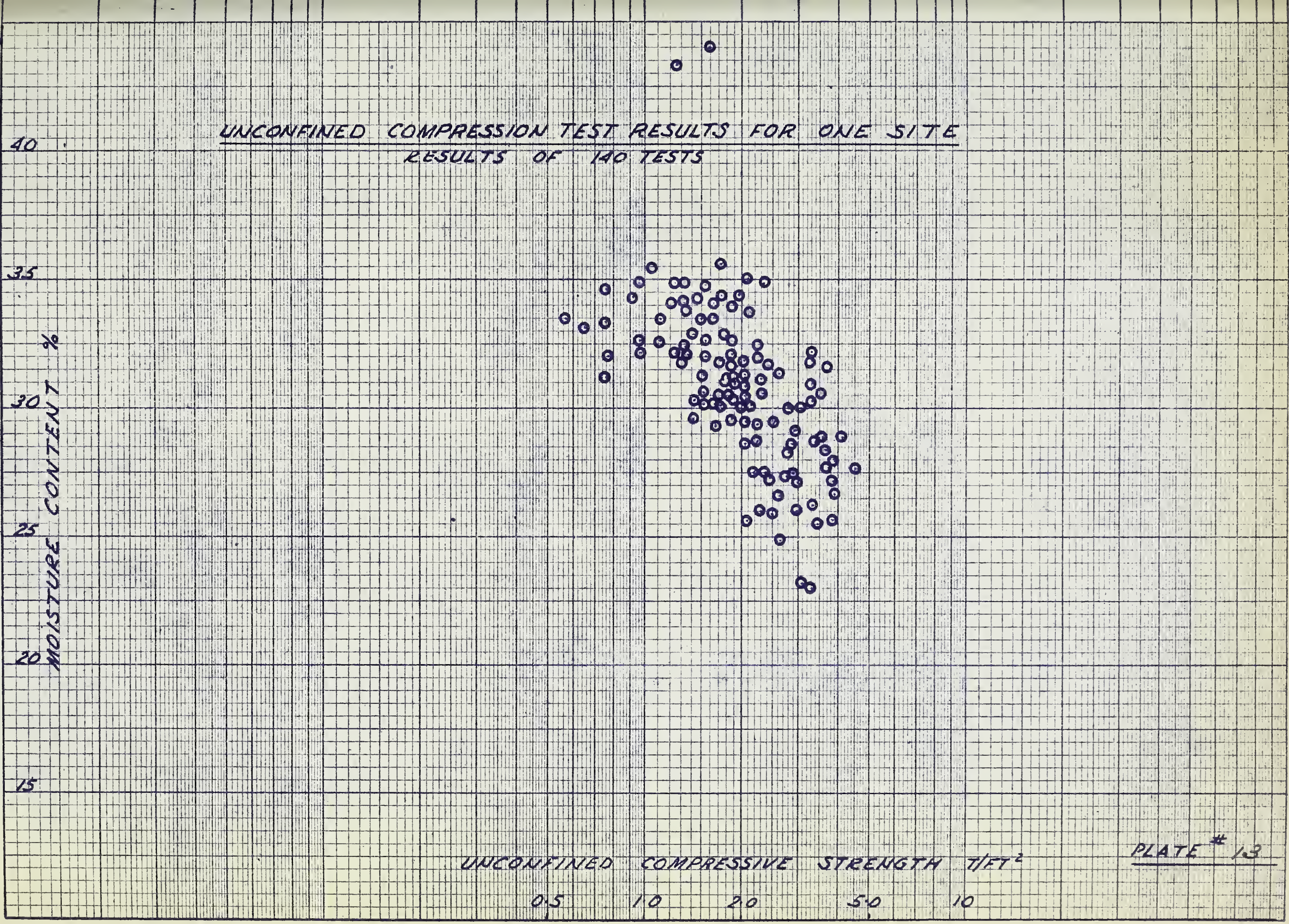
PLATE # 11

PLATE # 11


UNCONFINED COMPRESSION TEST RESULTS

SAMPLE OF TEST RESULTS SELECTED AT RANDOM

PLATE # 12


UNCONFINED COMPRESSION TEST RESULTS FOR ONE SITE

RESULTS OF 140 TESTS

PLATE # 13

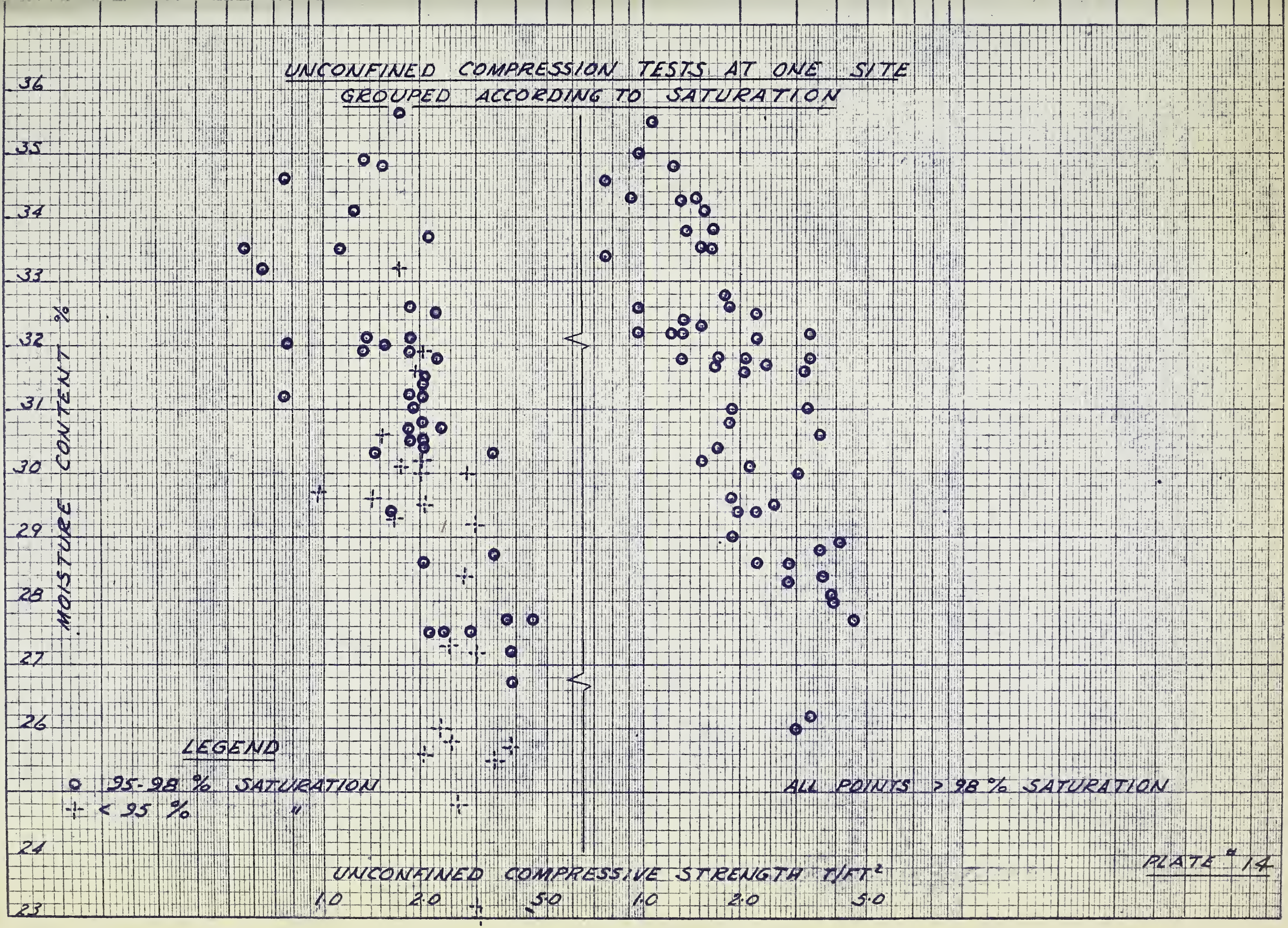
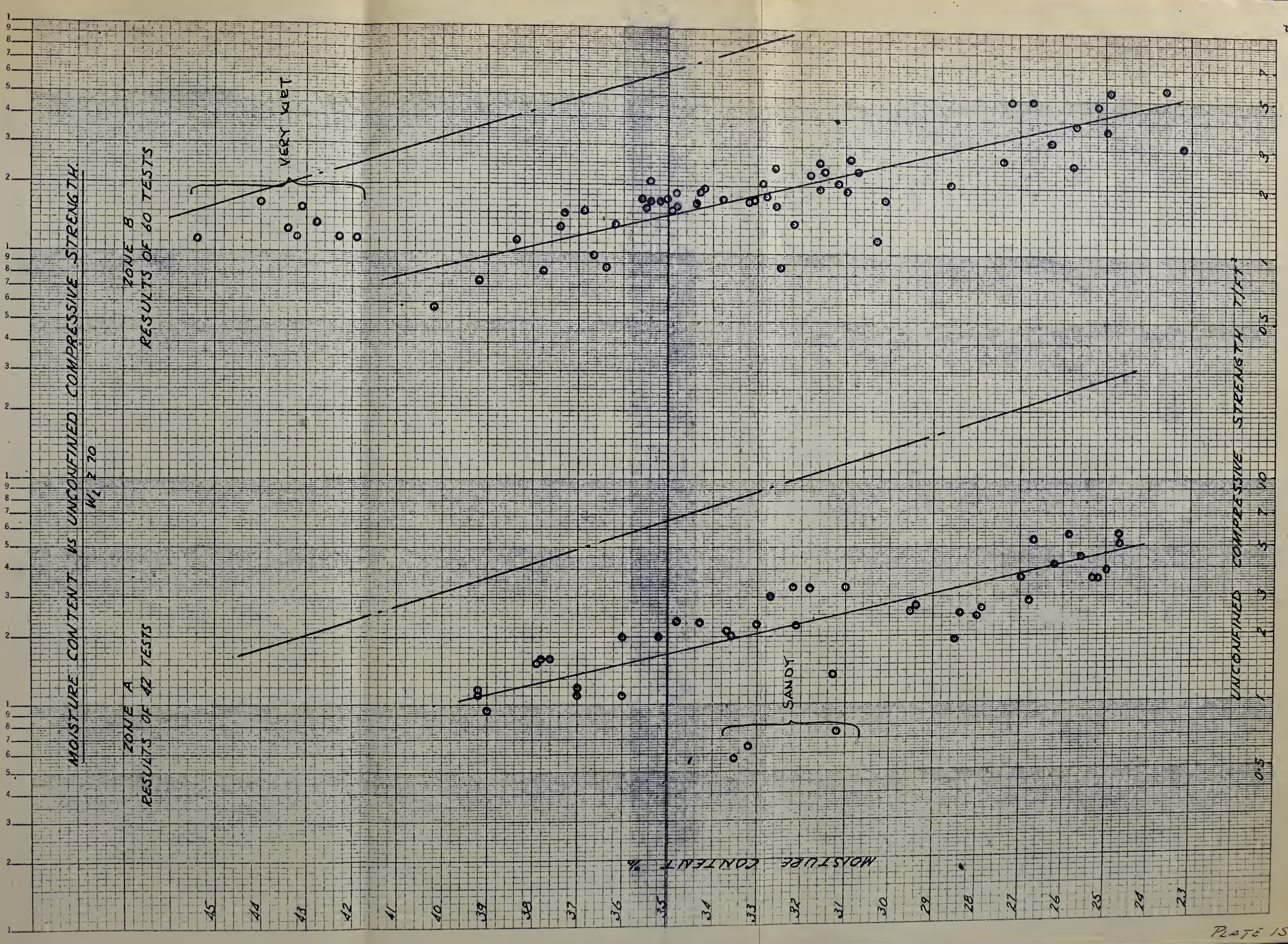

UNCONFINED COMPRESSION TESTS AT ONE SITE
GROUPED ACCORDING TO SATURATION
MOISTURE CONTENT %
36
35
34
33
32
31
30
29
28
27
26
24
23
LEGEND
o 95-98 % SATURATION
+ < 95 % "
ALL POINTS > 98 % SATURATION
UNCONFINED COMPRESSIVE STRENGTH T/FT²
1.0
2.0
5.0
1.0
2.0
5.0
PLATE # 14

PLATE # 14

PLATE # 15



KEUFFEL & ESSER CO., N. Y. NO. 359-83
Semi-Logarithmic, 4 Cycles × 10 to the inch.
MADE IN U.S.A.

PLATE 15

KEUFFEL & ESSER CO., N. Y. NO. 359-81
Semi-Logarithmic, 4 Cycles X 10 to the inch.
MADE IN U.S.A.

PLATE # 16

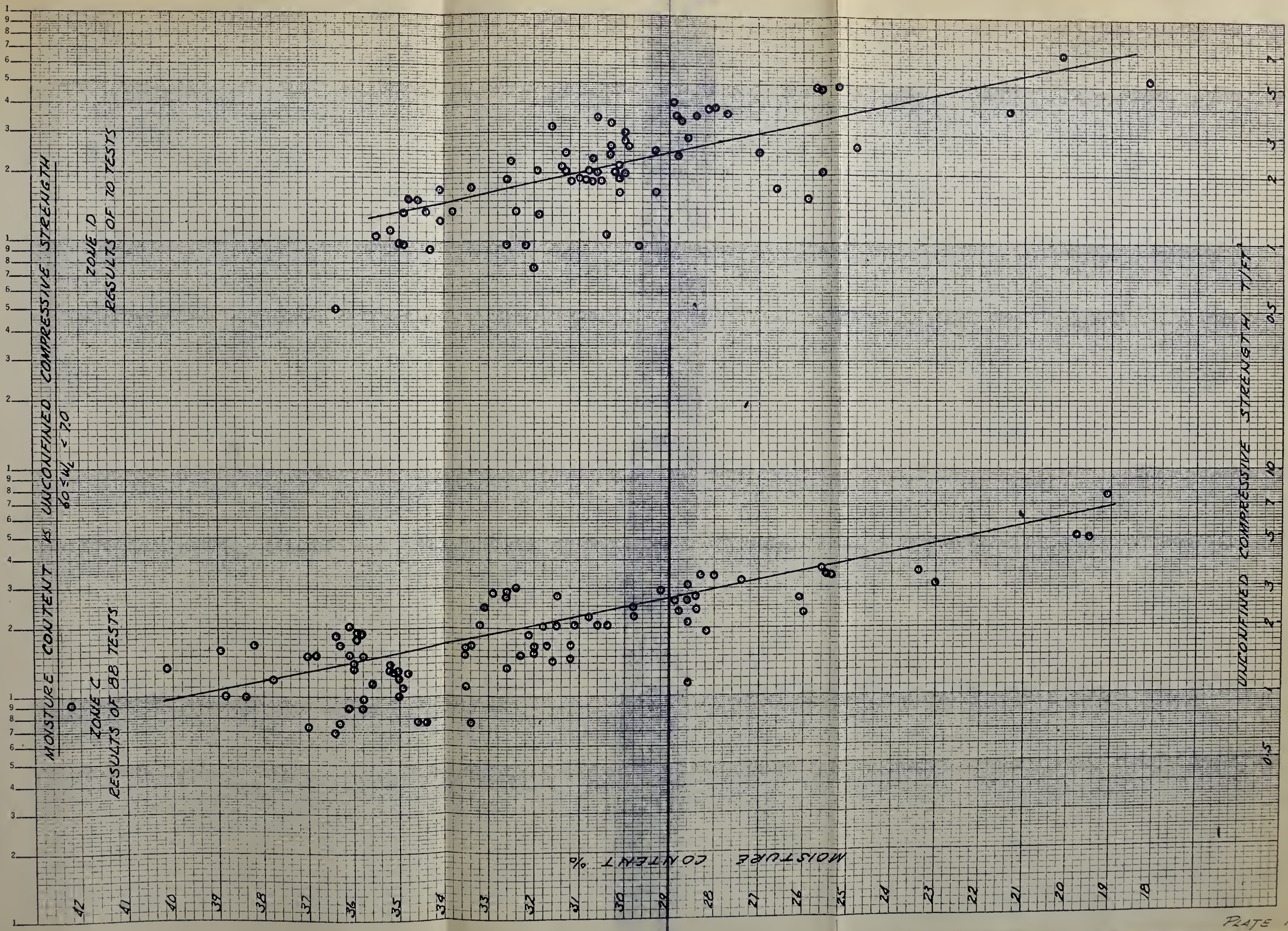


PLATE 16

KEUFFEL & ESSER CO., N. Y. NO. 359-81
Semi-Logarithmic, 1 Cycles × 10 to the inch
MADE IN U.S.A.

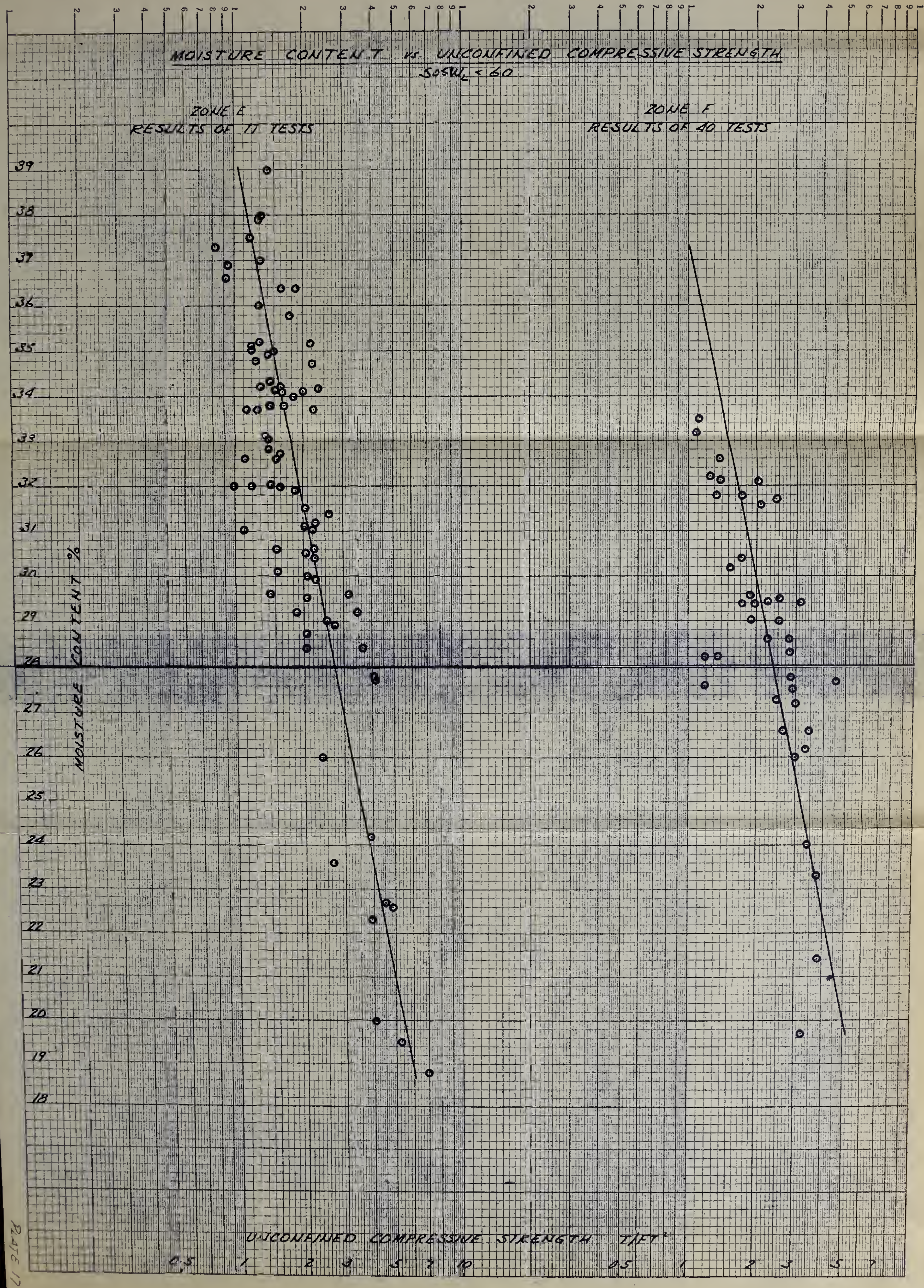


PLATE # 17

KEUFFEL & ESSER CO., N. Y. NO. 359-81
Semi-Logarithmic, 4 Cycles × 10 to the inch.
MADE IN U.S.A.

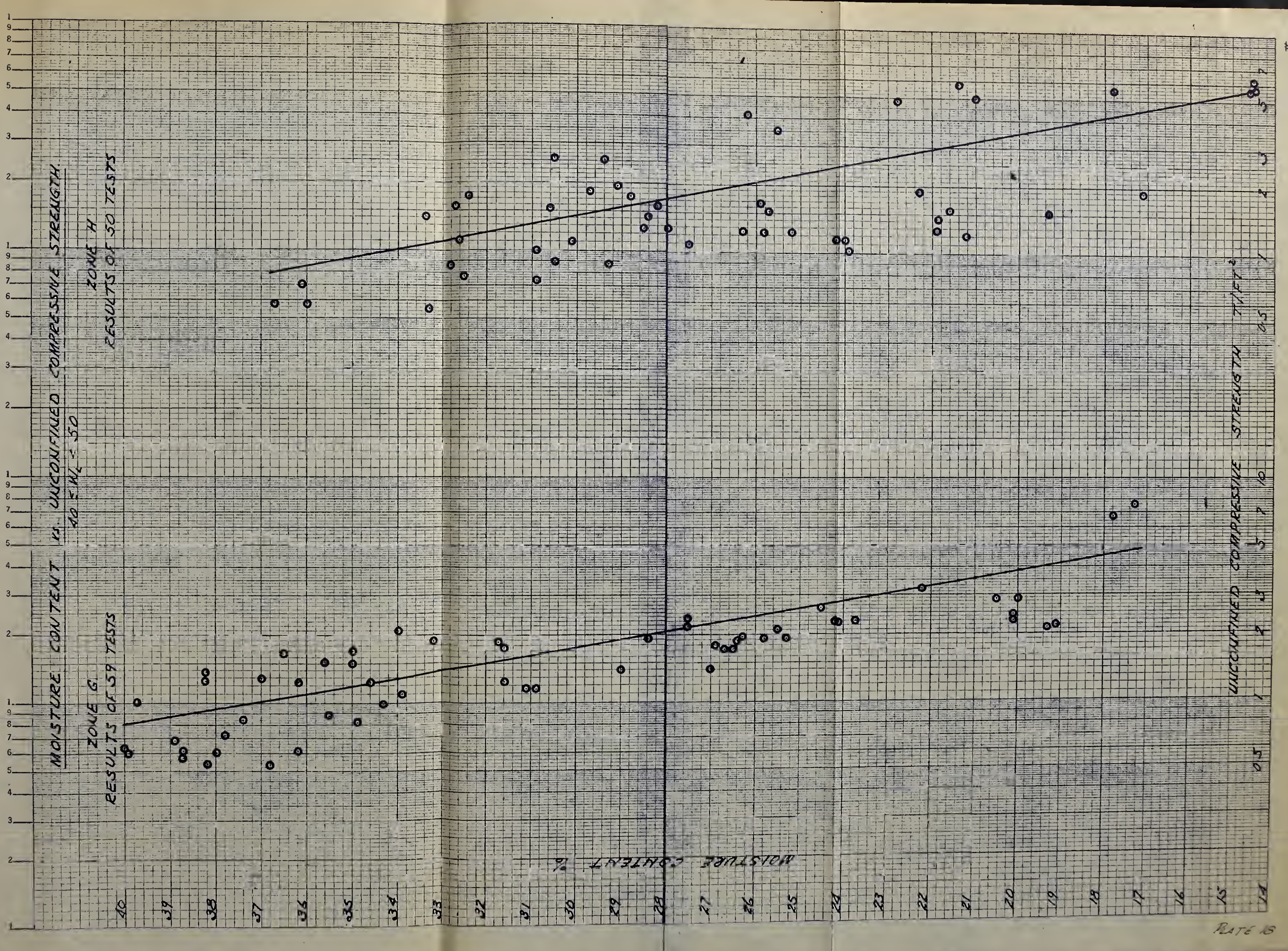


PLATE 18

PLATE # 19

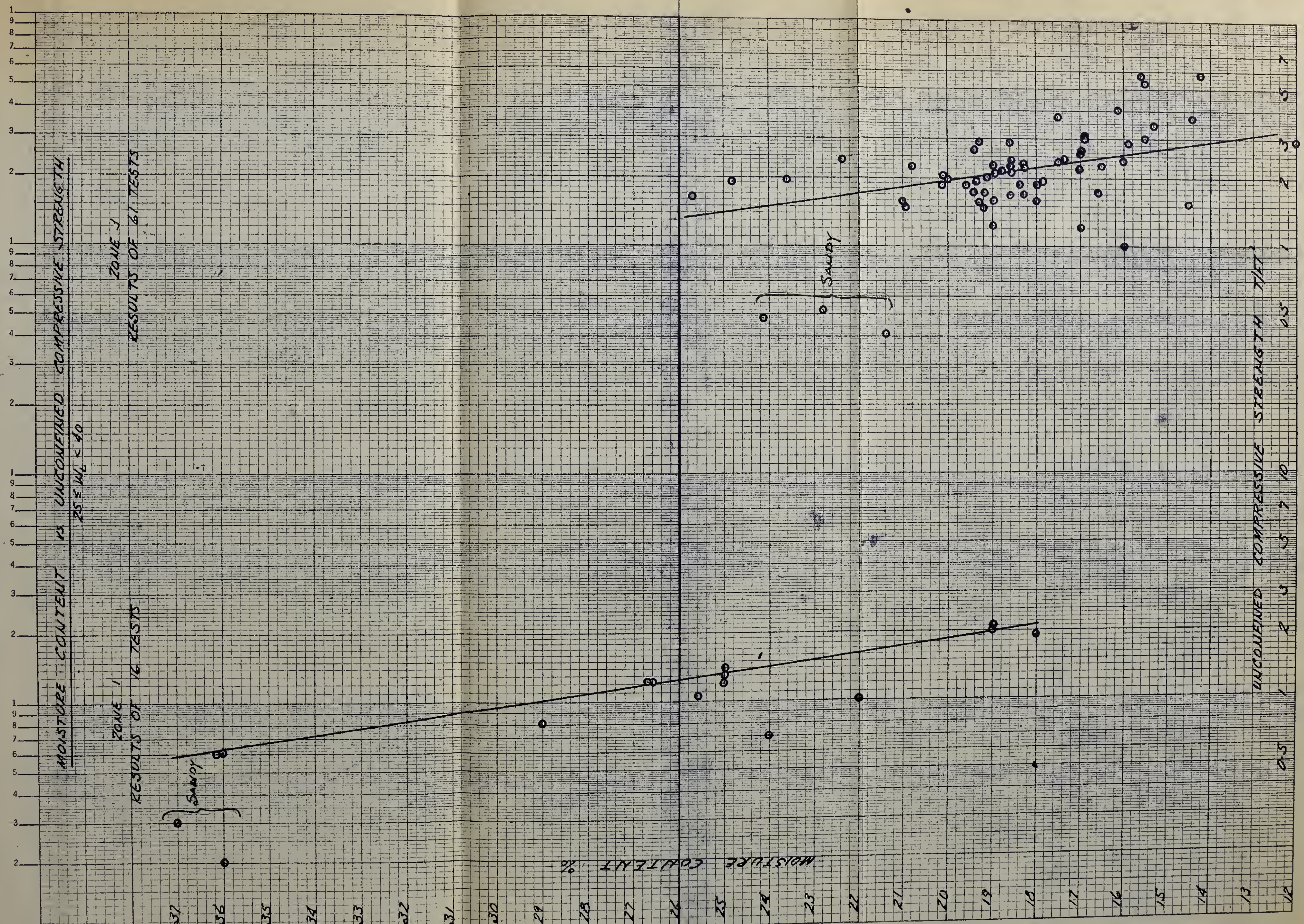
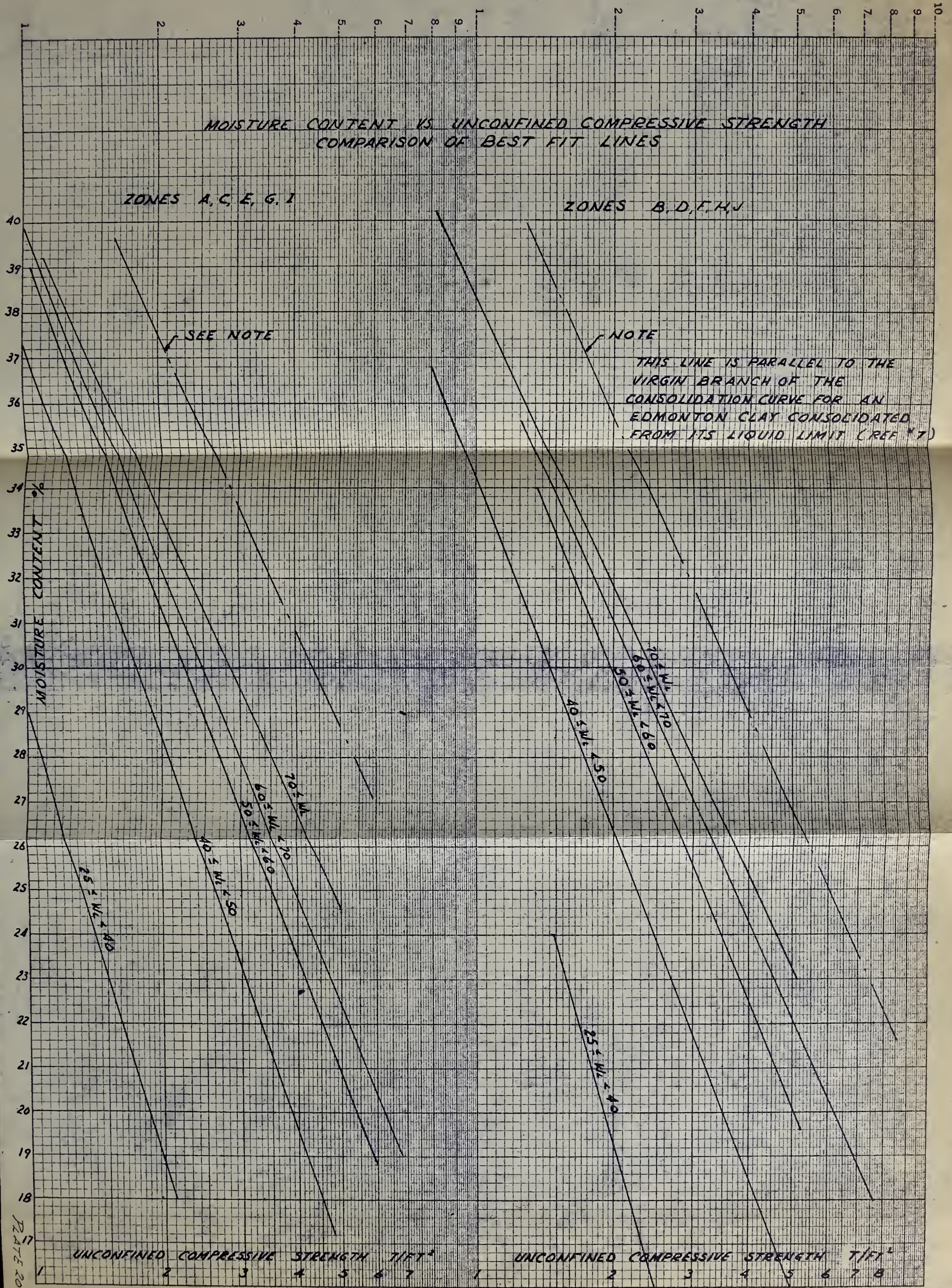


PLATE 19.

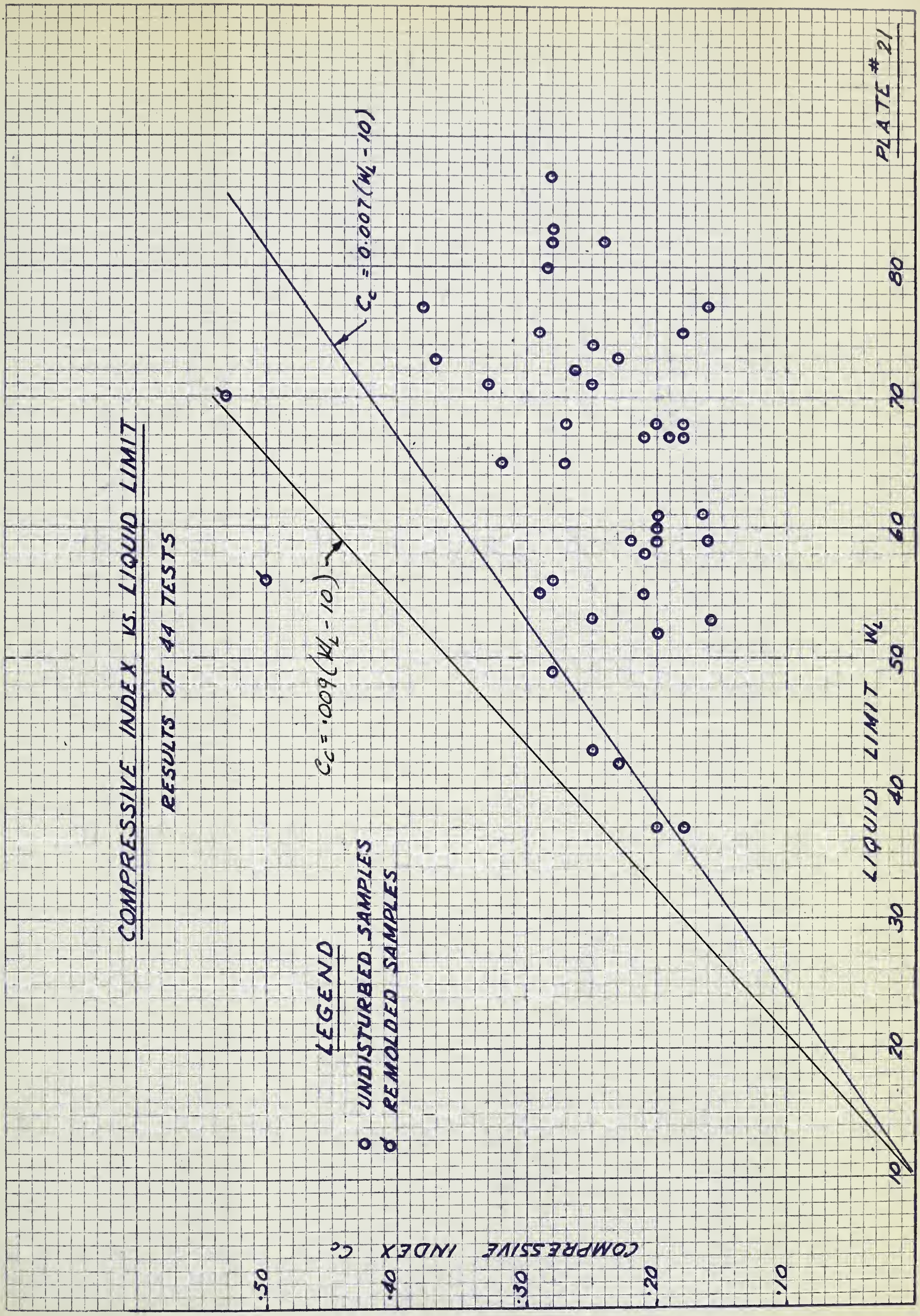
KEUFFEL & ESSER CO., N. Y. NO. 359-61
Semi-Logarithmic, 3 cycles × 10 to the inch, 5th lines accented.
MADE IN U. S. A.



MOISTURE CONTENT VS UNCONFINED COMPRESSIVE STRENGTH
COMPARISON OF BEST FIT LINES

PLATE 20

PLATE #2


COMPRESSIVE INDEX VS. LIQUID LIMIT

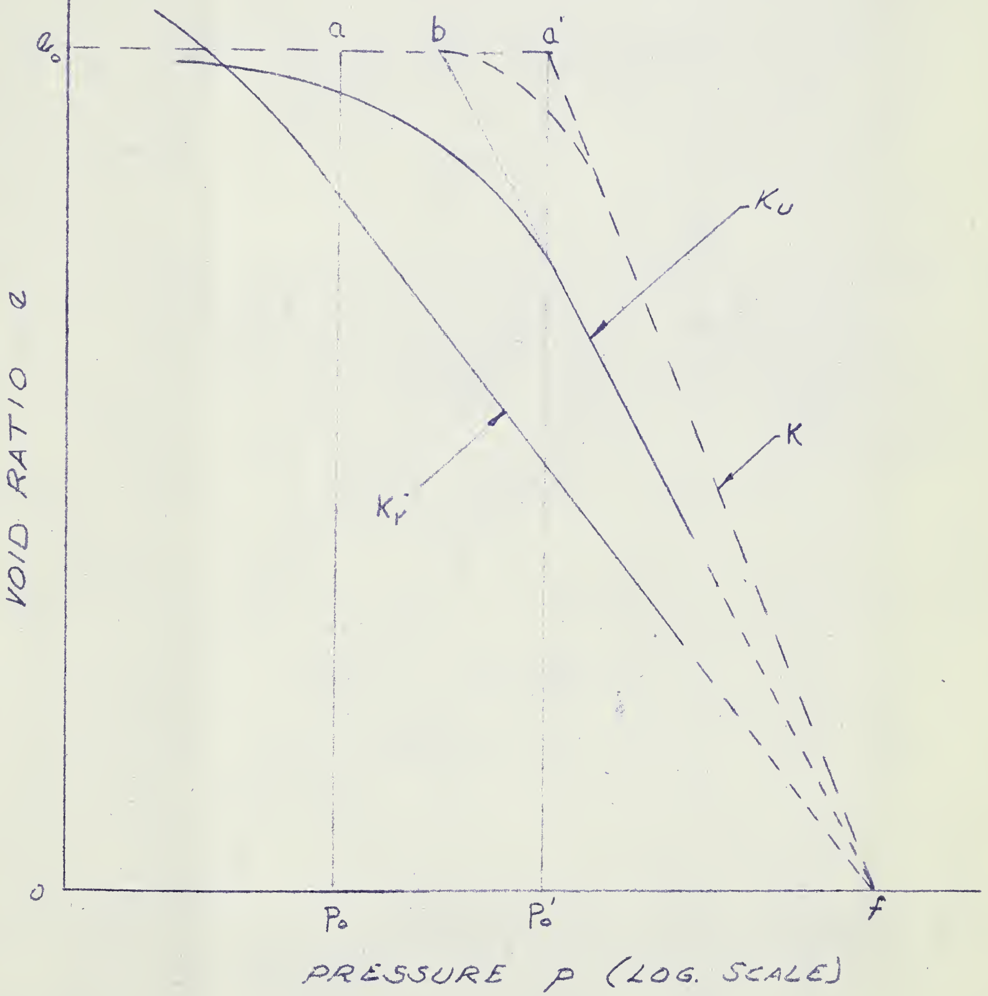
RESULTS OF 44 TESTS

LEGEND

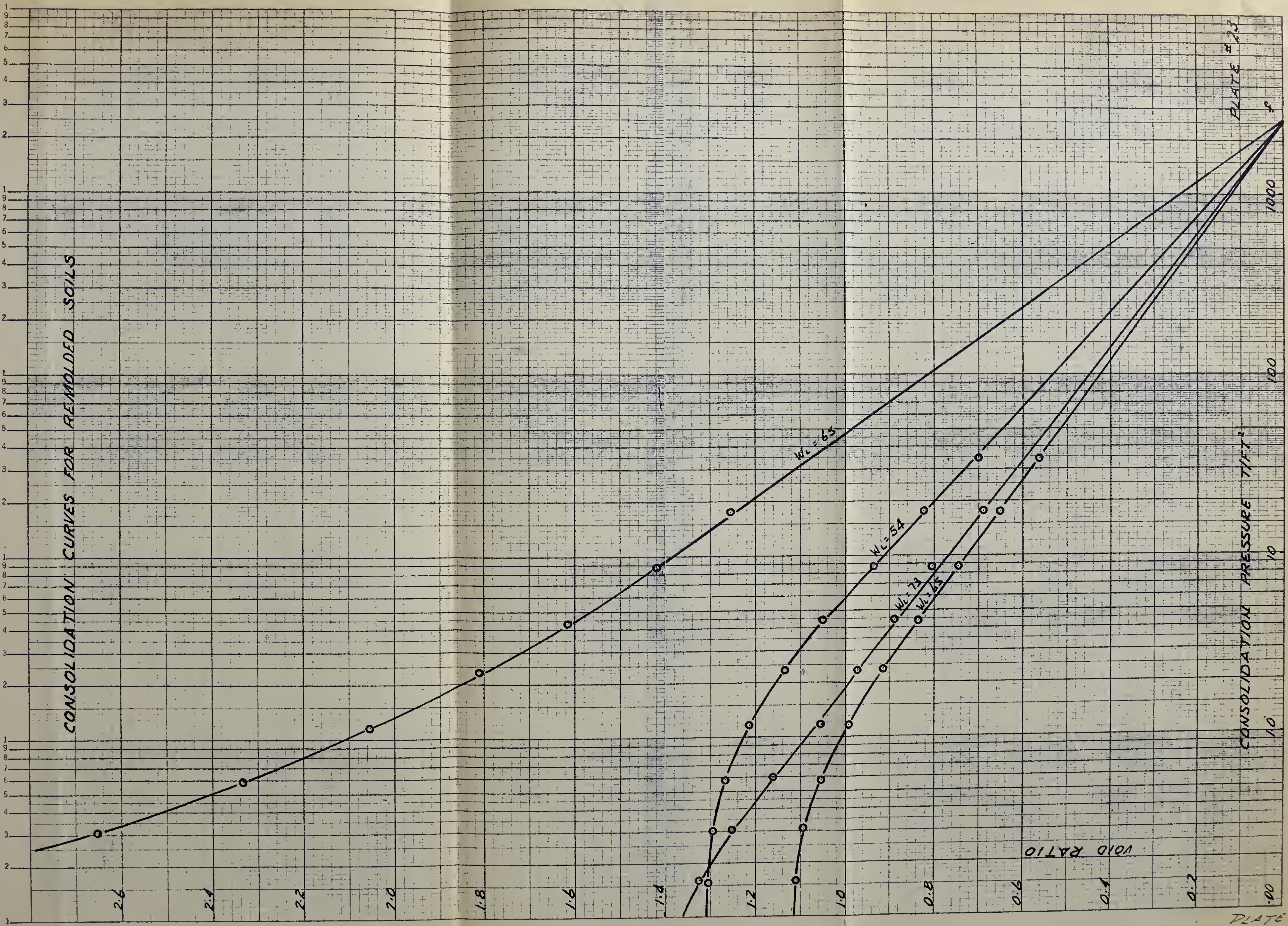
- o UNDISTURBED SAMPLES
- ø REMOLDED SAMPLES

TYPICAL e - log p CURVES

FOR UNDISTURBED (K_u) AND REMOLDED (K_r) SAMPLES OF PRELOADED CLAY OF ORDINARY SENSITIVITY



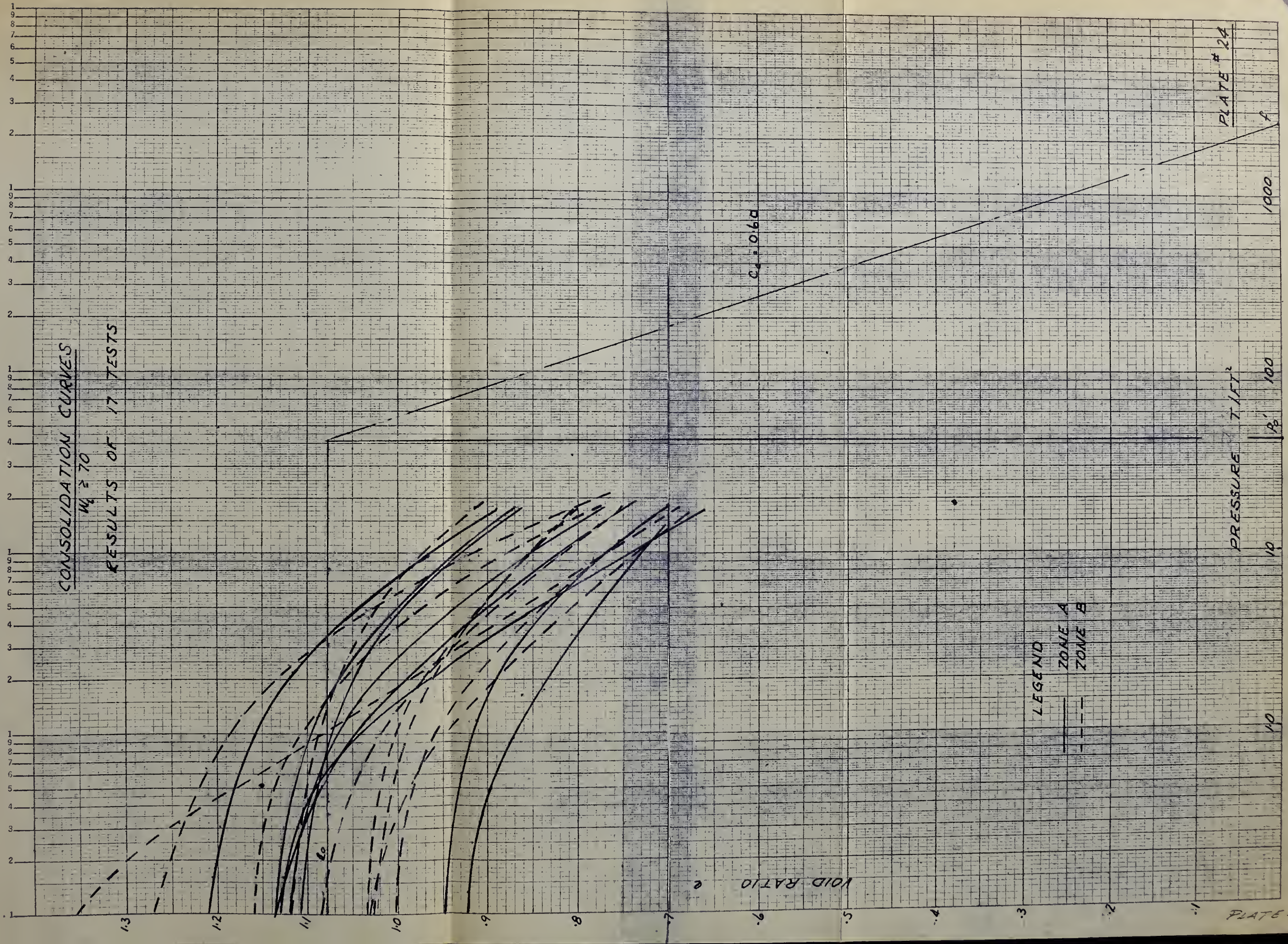
AFTER PECK, HANSON & THORNBURN



CONSOLIDATION CURVES FOR REMOLDED SOILS

359-91G KEUFFEL & ESSER CO.
Semi-Logarithmic, 5 Cycles × 10 to the inch.

PLATE 23


CONSOLIDATION CURVES
$W_L \geq 70$
RESULTS OF 17 TESTS
PLATE # 24
$C_c = .060$
PRESSURE T/FT²
p_o'
1000
100
10
1.0
1
9
8
7
6
5
4
3
2
LEGEND
ZONE A
ZONE B
e_o
VOID RATIO e
1.3
1.2
1.1
1.0
.9
.8
.7
.6
.5
.4
.3
.2
.1
KEUFFEL & ESSER CO., N. Y. NO. 359-91G
Semi-Logarithmic, 3 Cycles × 10 to the inch, 5th lines accented.
MADE IN U.S.A.
PLATE 24

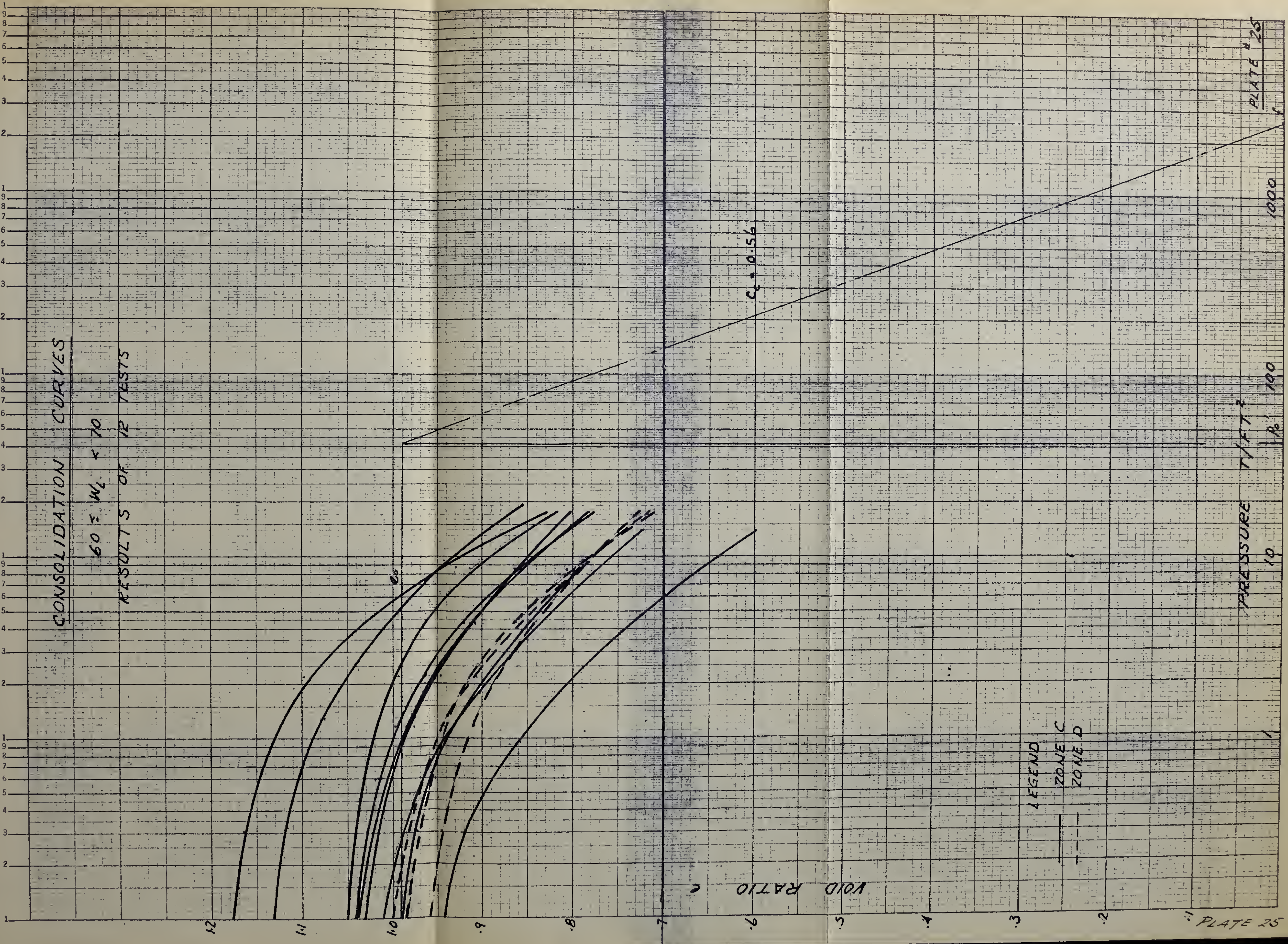


PLATE 25

359-91G KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles × 10 to the inch
MADE IN U.S.A.

359-91G KEUFFEL & ESSER CO.
Semi-Logarithmic, 5 Cycles × 10 to the inch

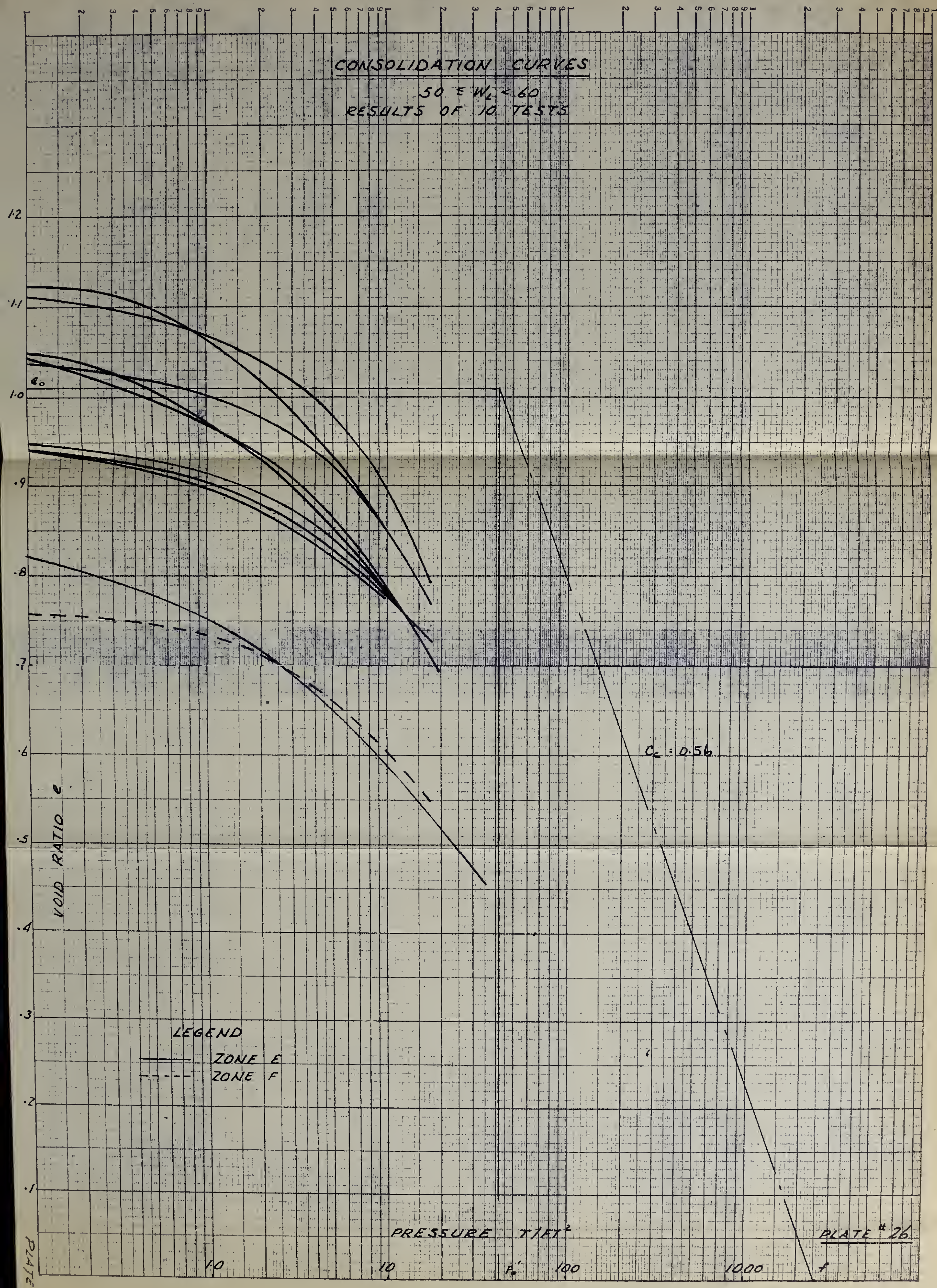


PLATE 26

359-91G KEUFFEL & ESSER CO.
Semi-Logarithmic, 5 Cycles × 10 to the inch
MADE IN U.S.A.

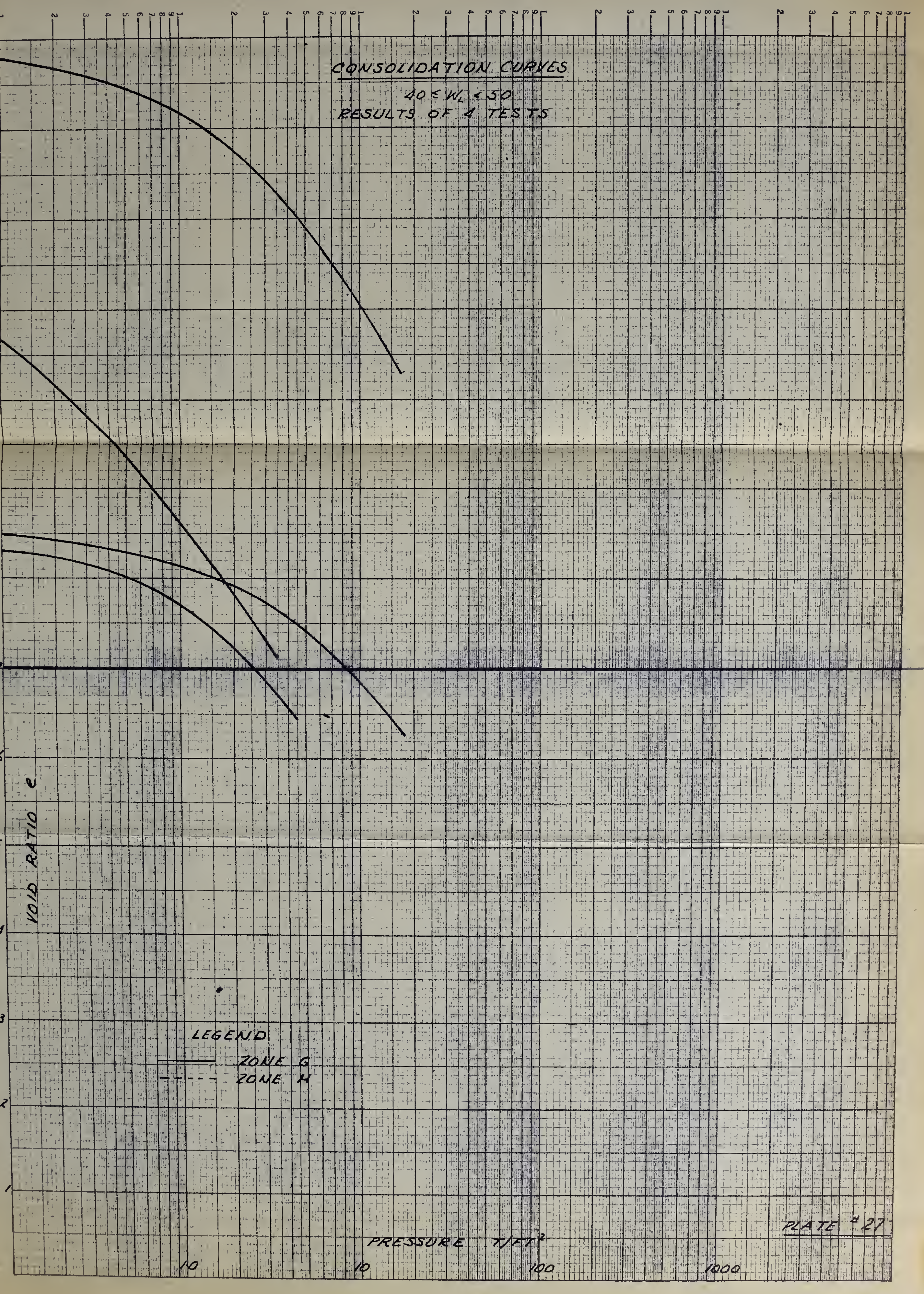


PLATE 27

359-91G KEUFFEL & ESSER CO.
Semi-Logarithmic, 5 Cycles × 10 to the inch.
MADE IN U.S.A.

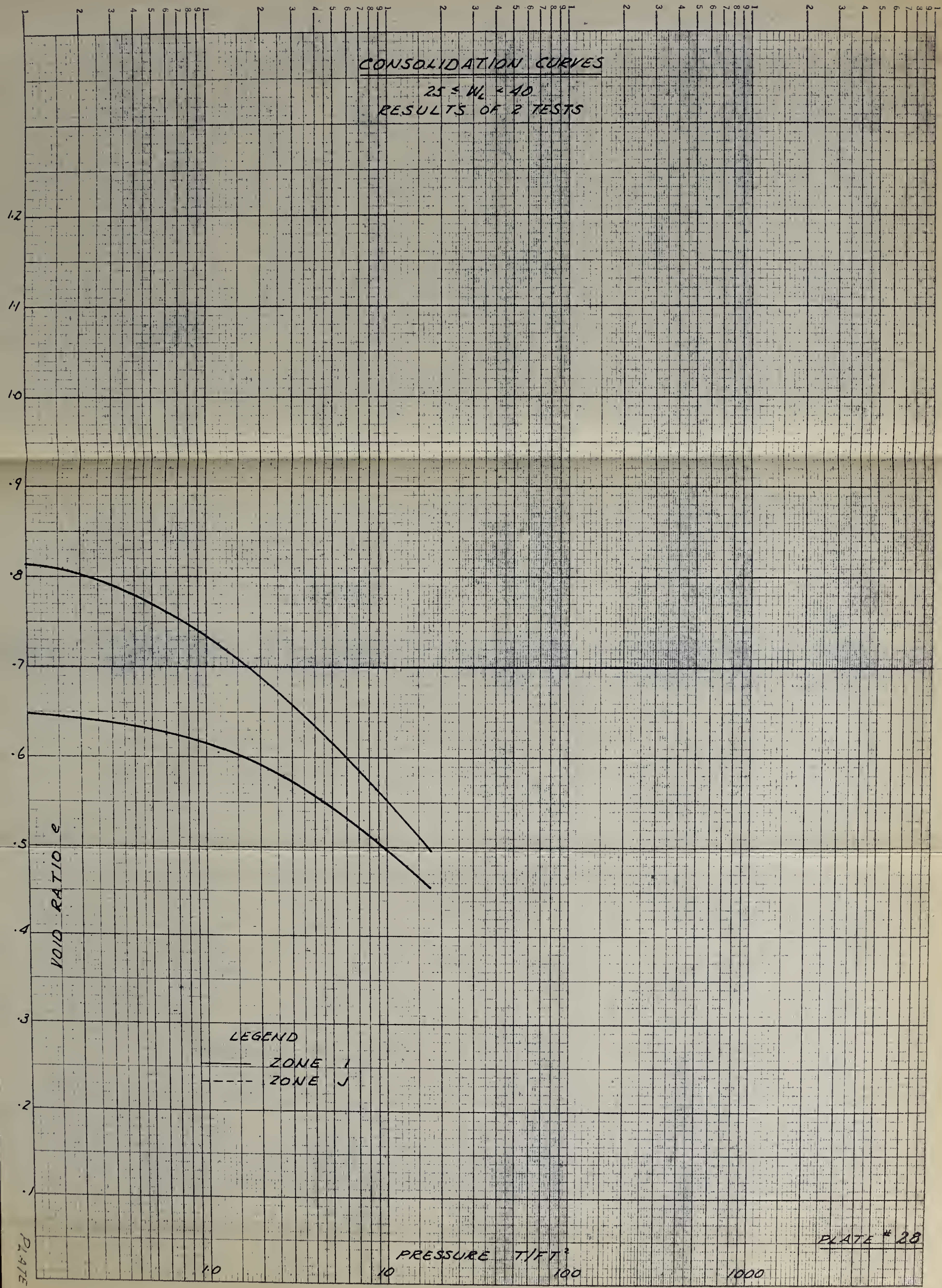
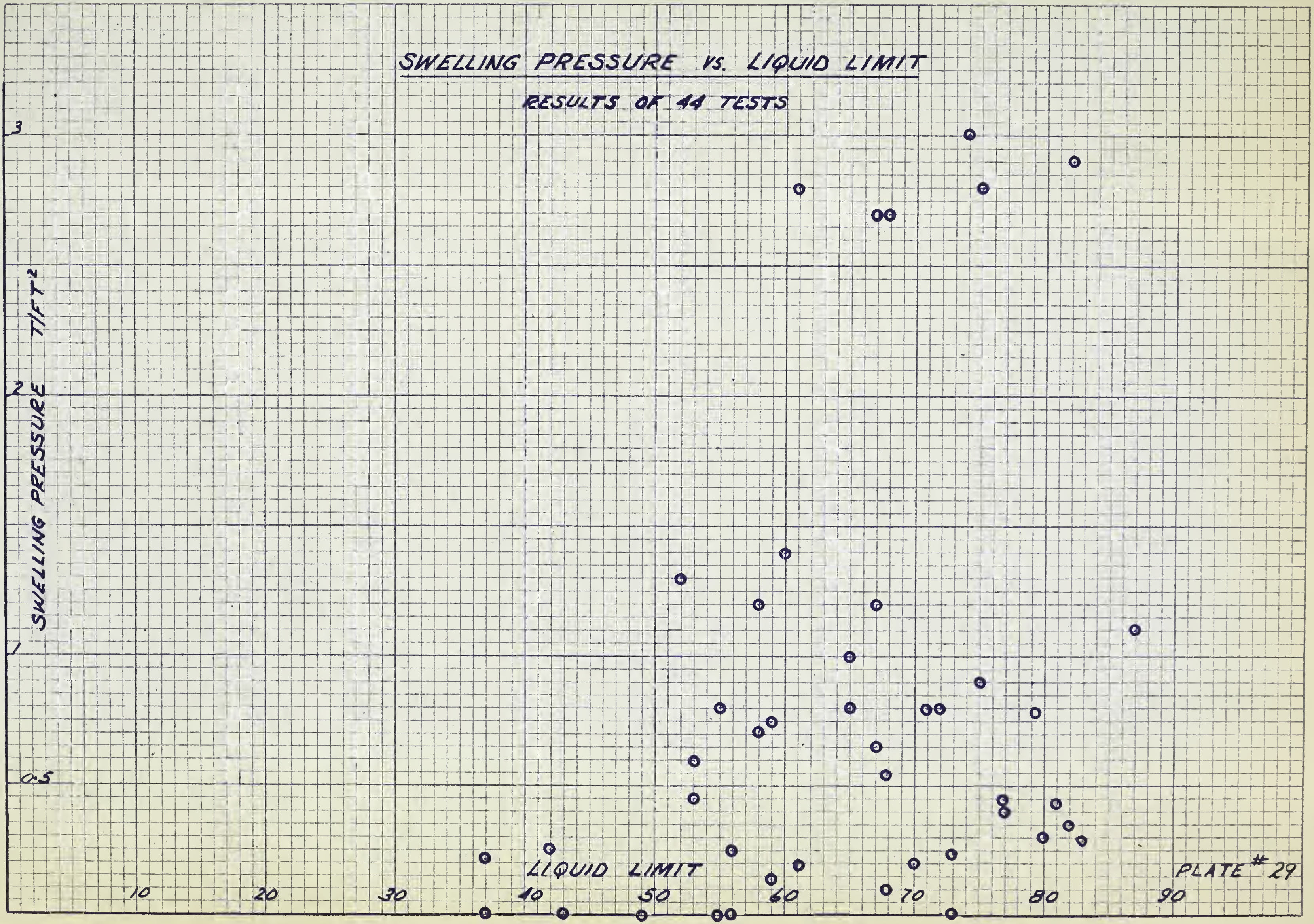


PLATE 28



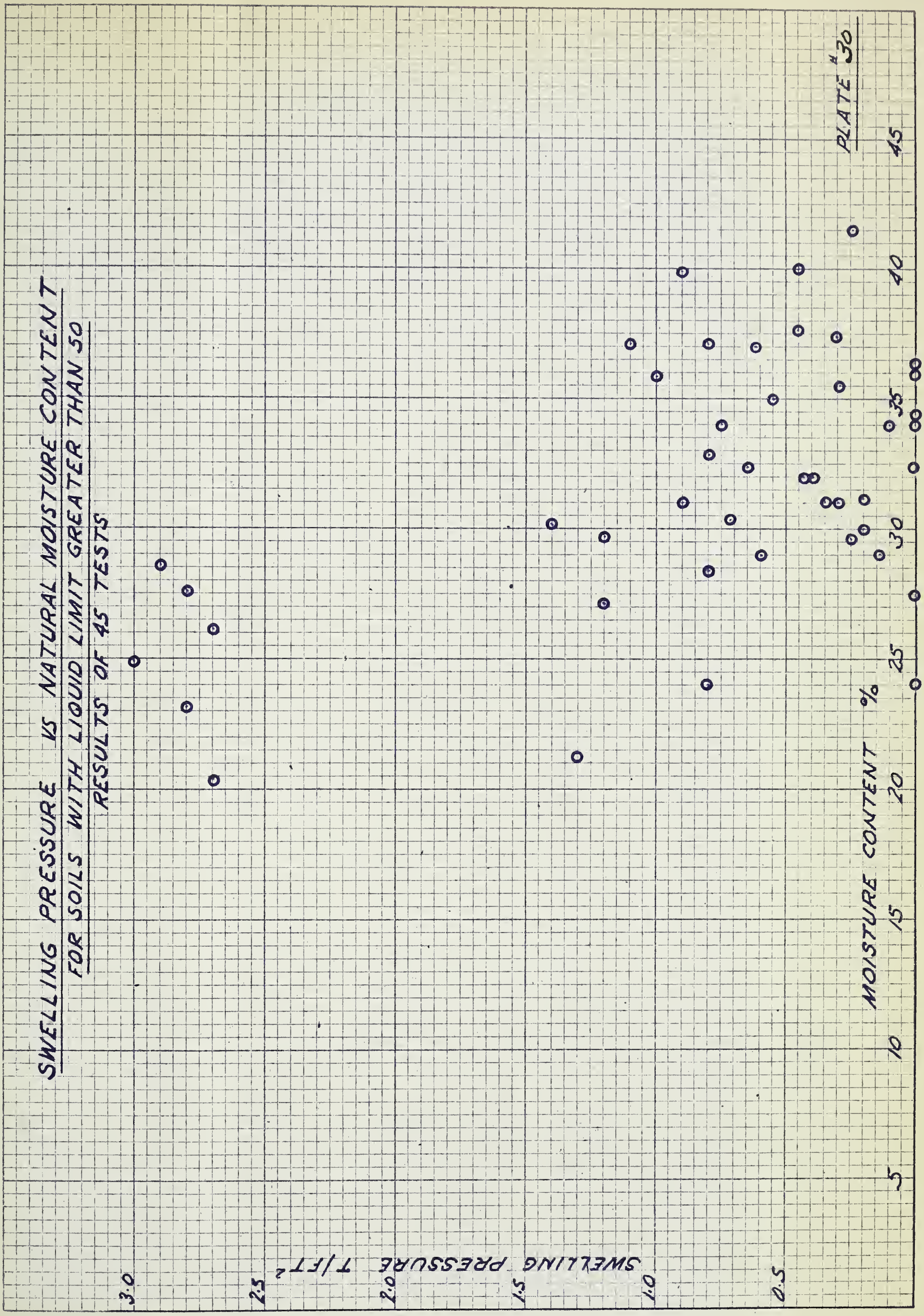
SWELLING PRESSURE VS. LIQUID LIMIT

RESULTS OF 44 TESTS

PLATE # 29

PLATE

PLATE # 30



SWELLING PRESSURE VS NATURAL MOISTURE CONTENT
FOR SOILS WITH LIQUID LIMIT GREATER THAN 50
RESULTS OF 45 TESTS

SWELLING PRESSURE VS LIQUIDITY INDEX

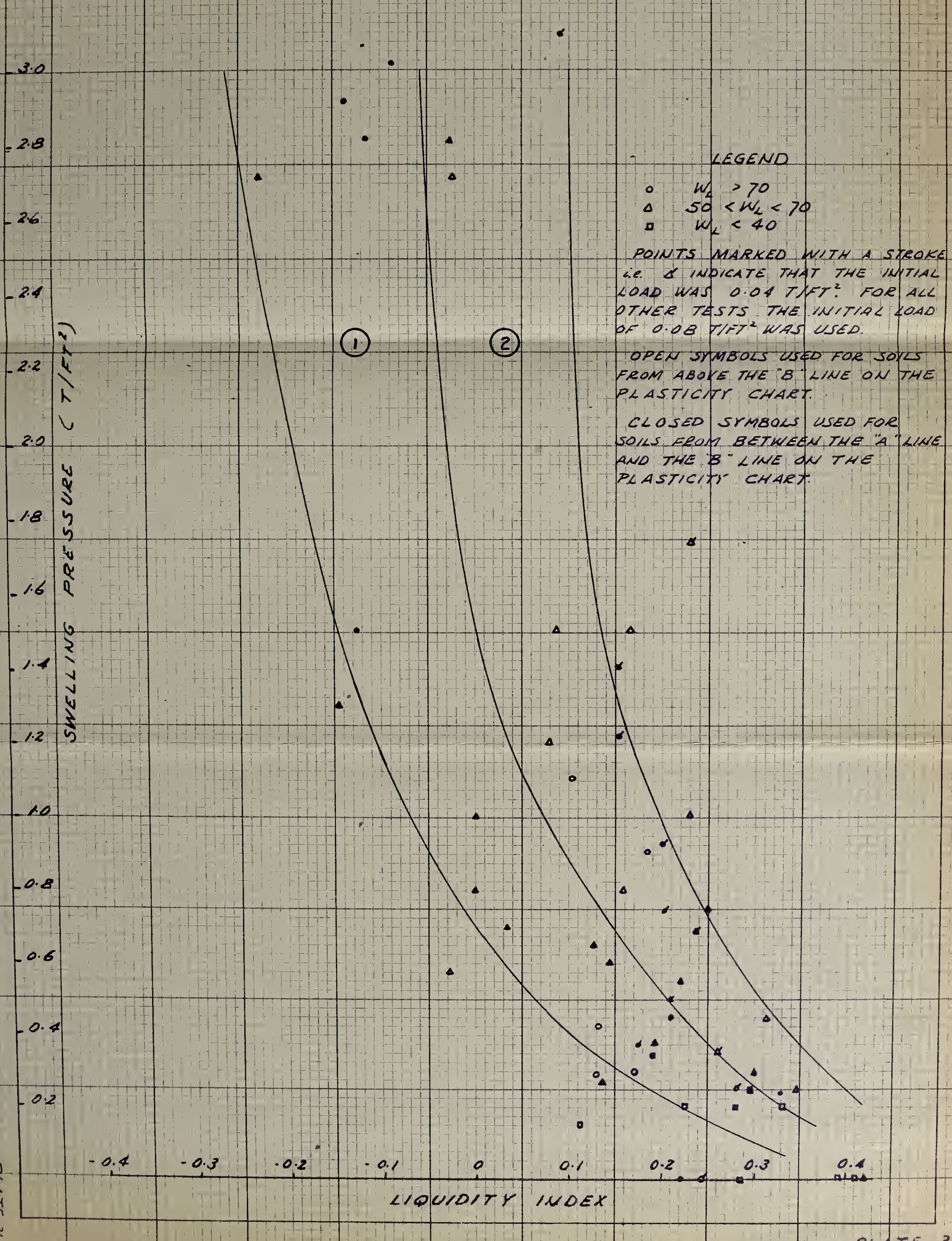
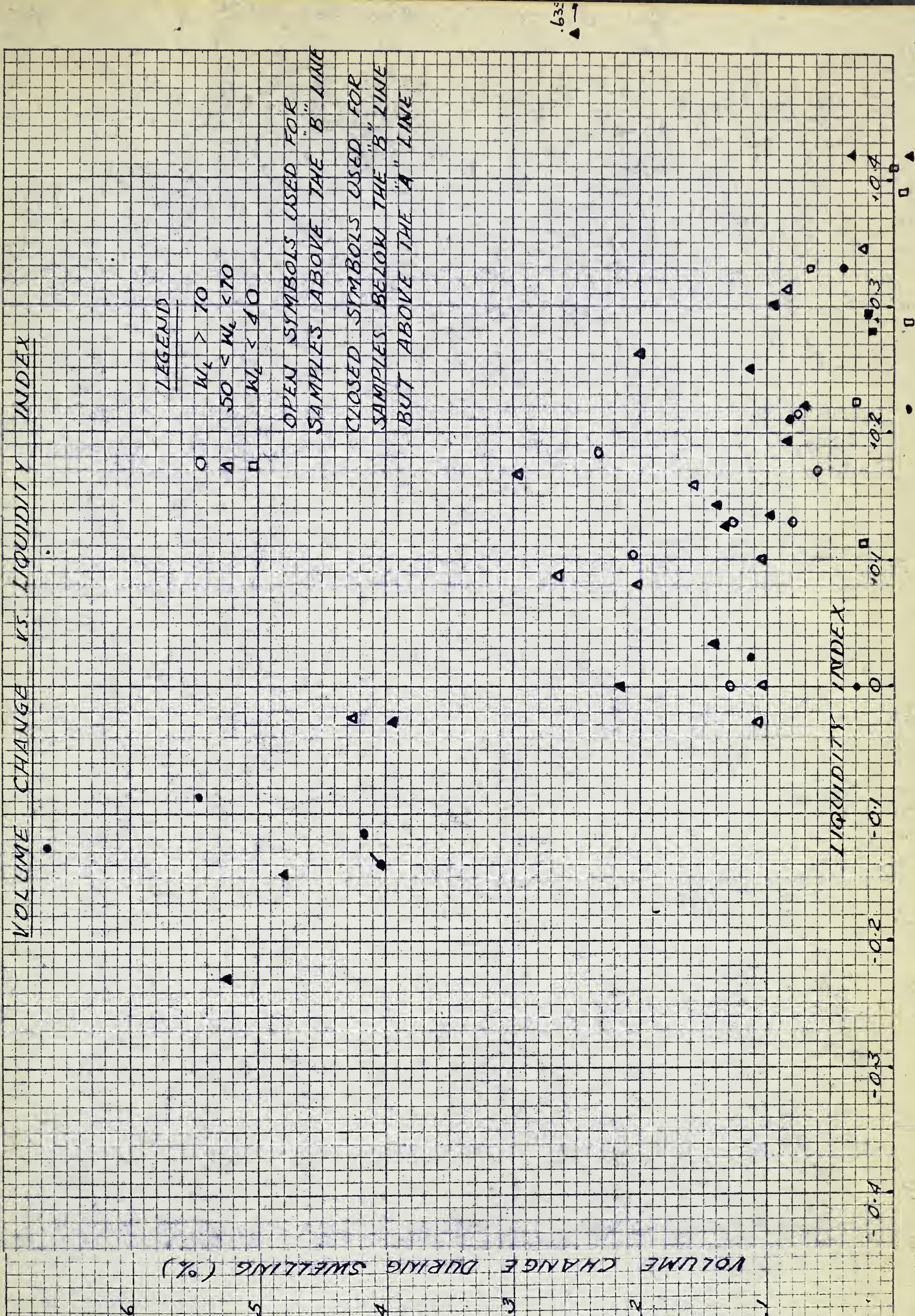


PLATE 31



VOLUME CHANGE VS. LIQUIDITY INDEX

PLATE #32

PLATE # 32

LOAD-DEFORMATION CURVES FOR SOILS IN CONFINED COMPRESSION

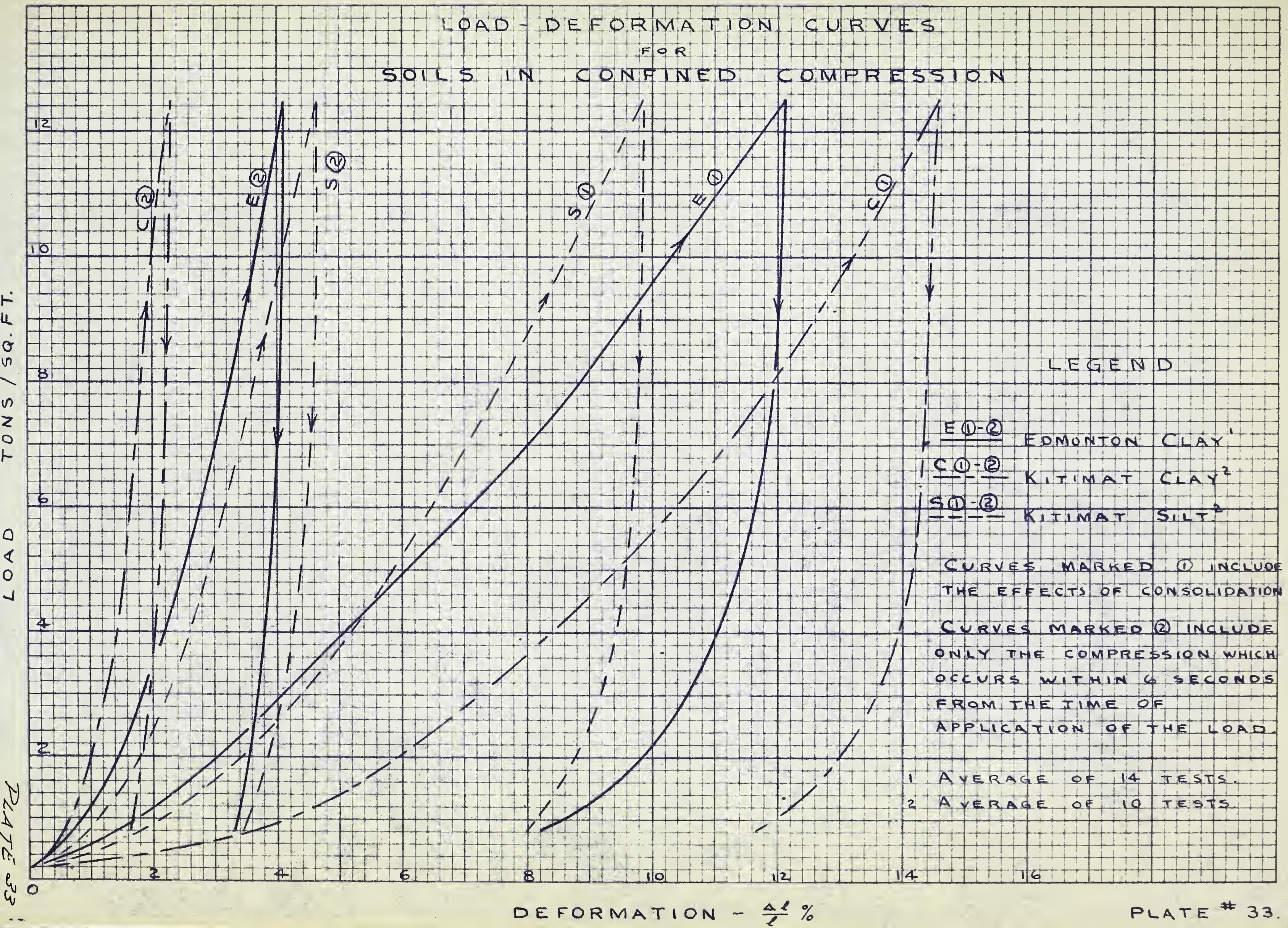


PLATE # 33.

PLATE 33

TABLE V

LOAD - DEFORMATION CHARACTERISTICS IN CONFINED COMPRESSION

	EDMONTON CLAY[1]		KITIMAT CLAY[2]		KITIMAT SILT[2]	
LOAD INCREMENT TONS/SQ. FT.	DEFORMATION % OF ORIGINAL HT.	% OF DEFORMATION WITHIN 6 SECS	DEFORMATION % OF ORIGINAL HT	% OF DEFORMATION WITHIN 6 SECS	DEFORMATION % OF ORIGINAL HT.	% OF DEFORMATION WITHIN 6 SECS
·10 to ·20	0.1	60	0.5	37	0.3	56
·20 to ·37	0.5	42	1.0	10	0.5	52
·37 to ·68	1.0	41	1.7	16	0.9	53
·68 to 1.55	1.4	37	2.1	15	1.1	54
1.55 to 3.10	2.0	33	2.6	14	1.4	48
3.10 to 6.26	3.2	30	3.0	14	1.8	42
6.26 to 12.49	4.8	26	3.7	17	2.8	44
12.49 to 6.36	- 0.3	44	- 0.3	47	- 0.3	88
6.36 to 3.18	- 1.1	24	- 0.6	31	- 0.7	76
3.18 to ·82	- 2.6	14	- 2.0	15	- 1.3	55

1. AVERAGE OF 14 TESTS
2. " " 10 "

UNCONFINED COMPRESSIVE STRENGTH VS BLOW COUNT AT 0·8 FT. PENETRATION

RESULTS OF 81 TESTS

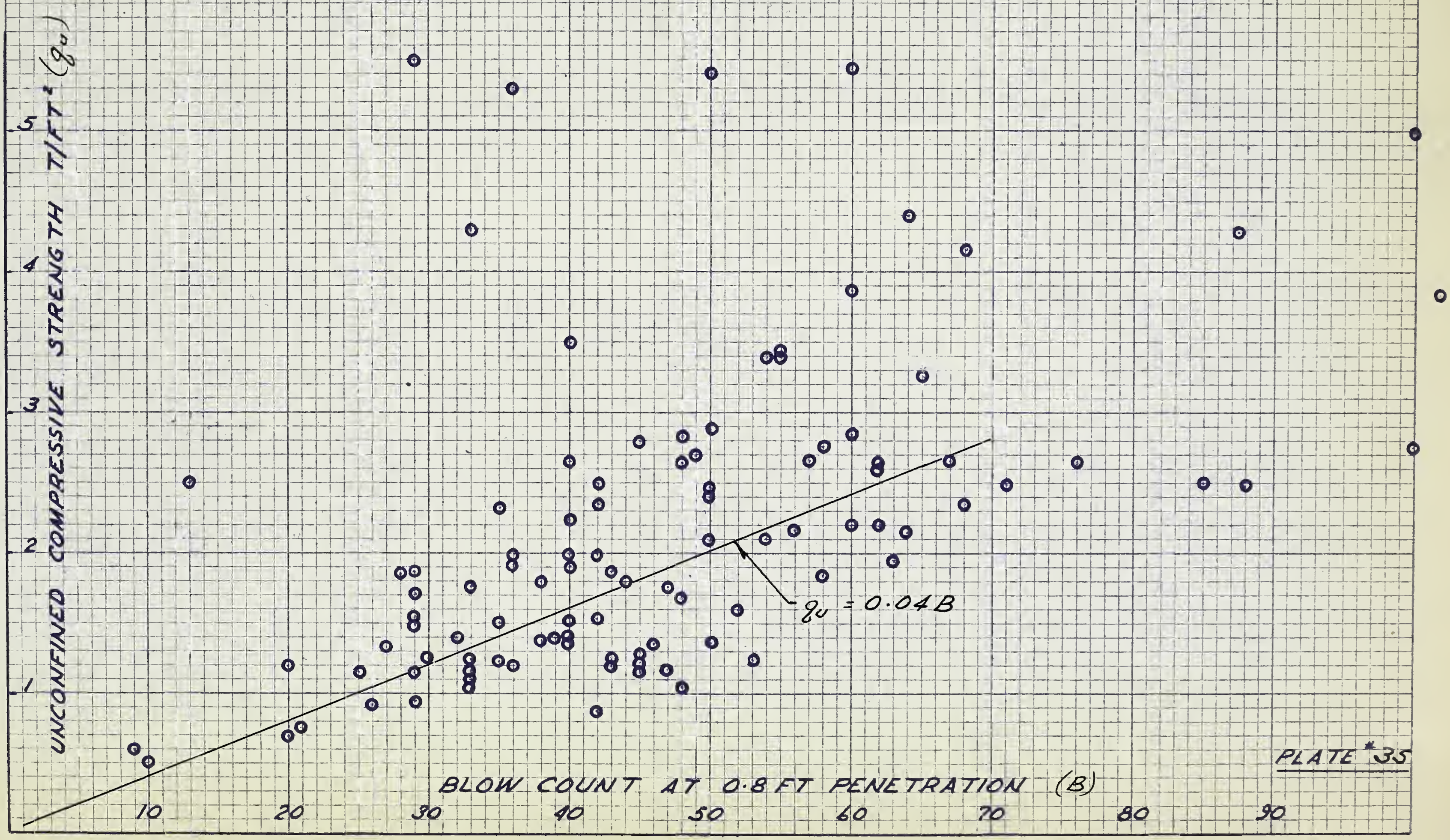


PLATE * 35

PLATE * 3

UNCONFINED COMPRESSIVE STRENGTH vs. BLOW COUNT AT 0·8FT PENETRATION

FOR SOILS WITH LIQUID LIMIT LESS THAN 50

RESULTS OF 43 TESTS

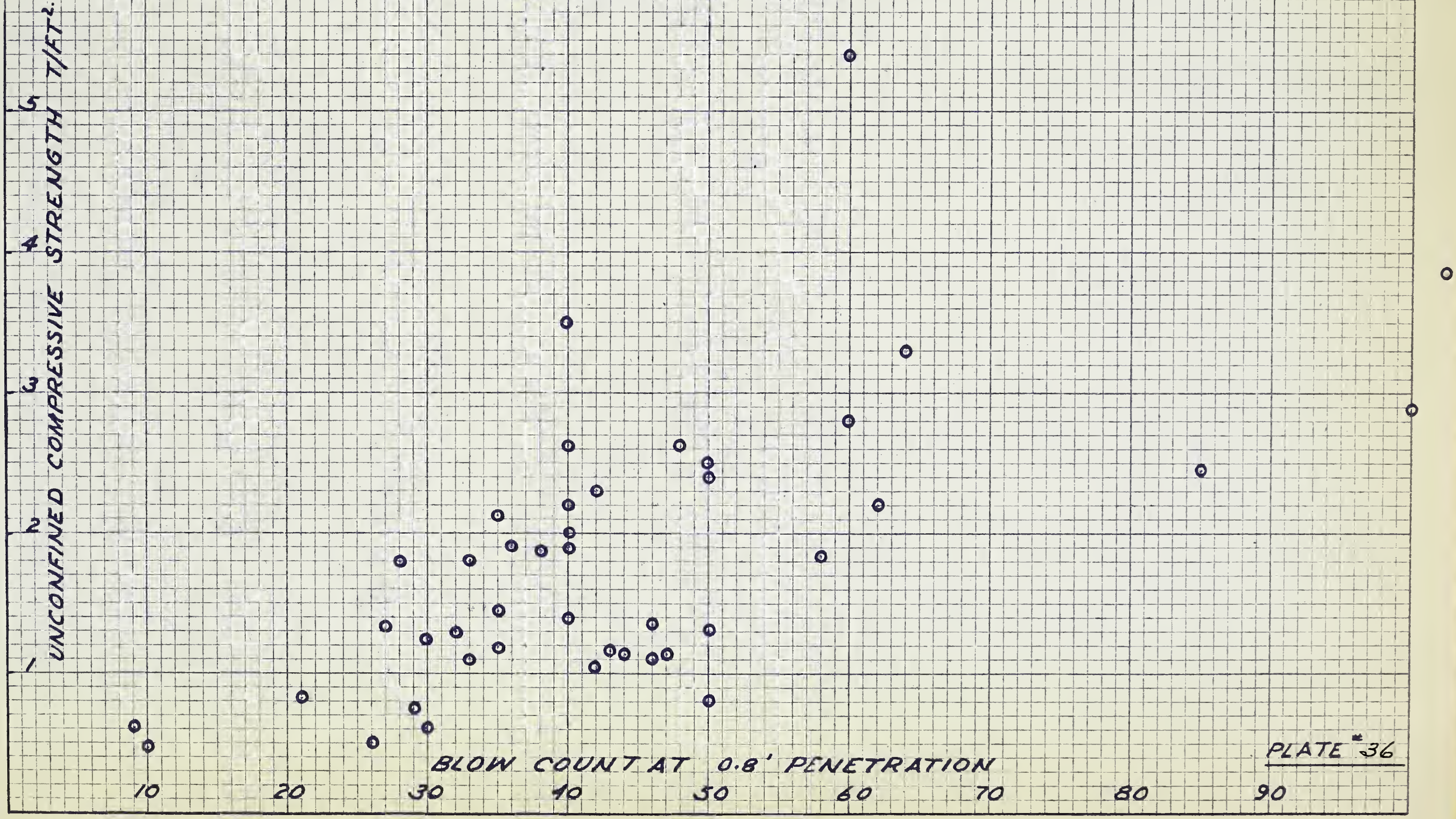
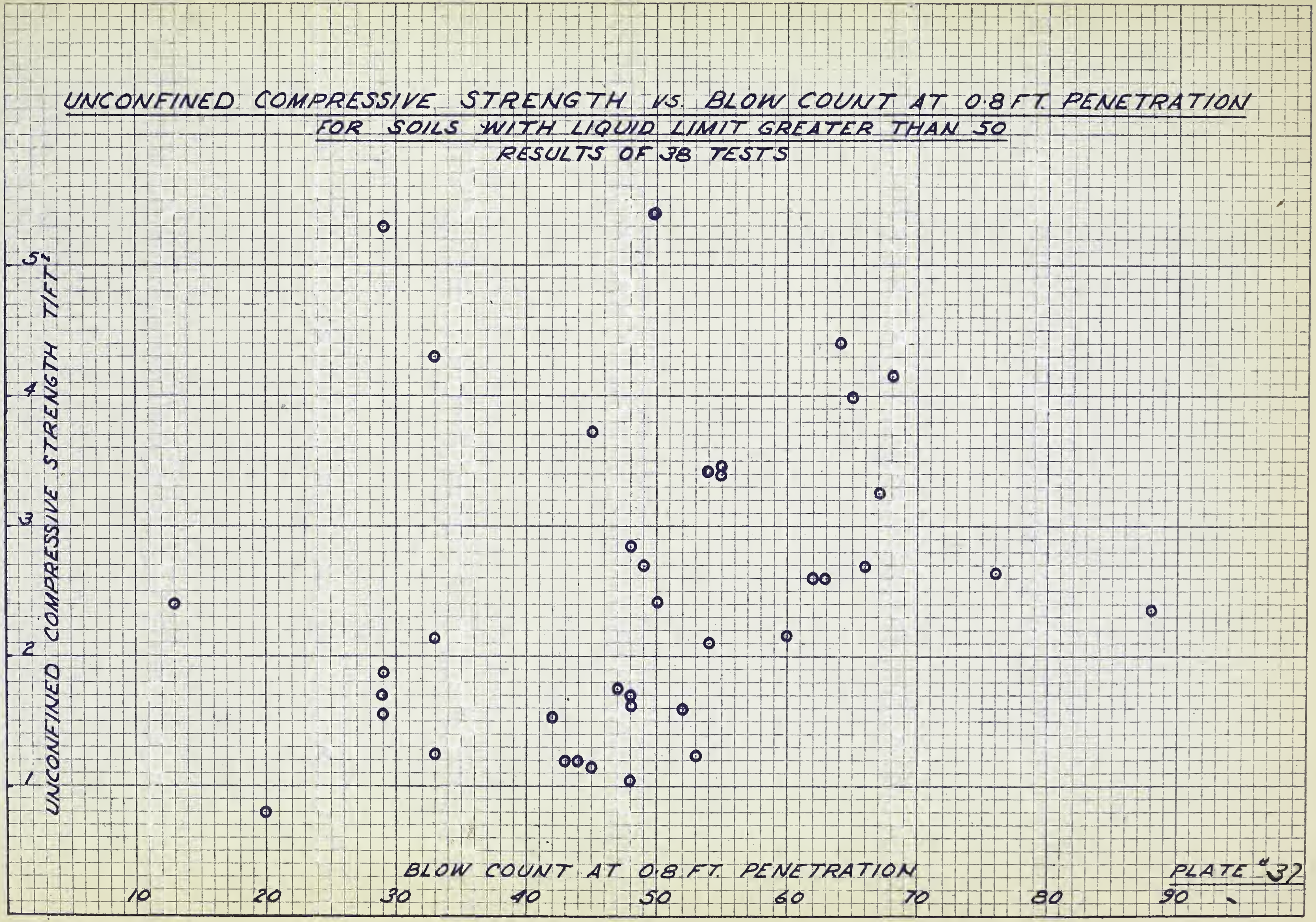


PLATE #36



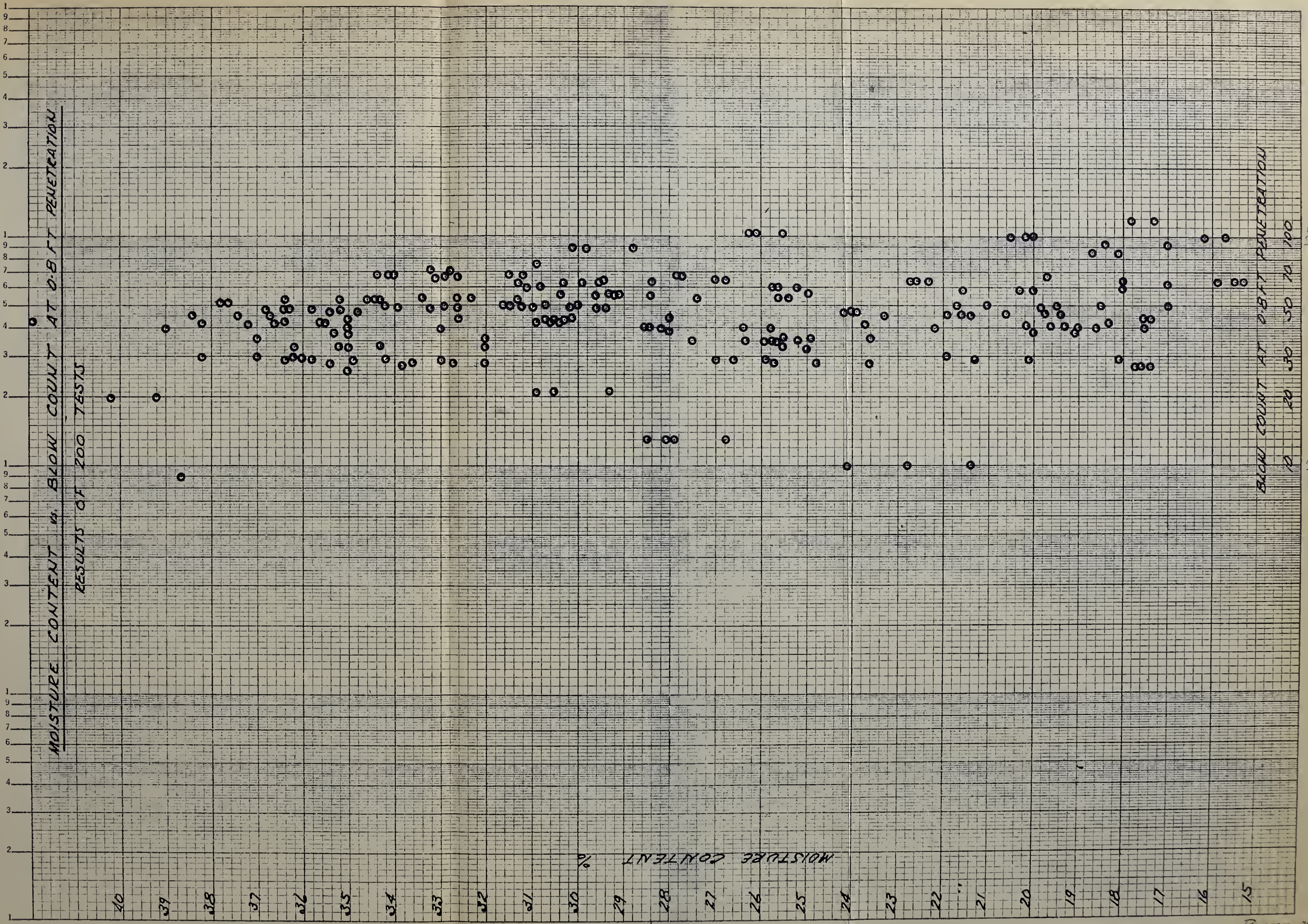
UNCONFINED COMPRESSIVE STRENGTH VS. BLOW COUNT AT 0.8 FT. PENETRATION

FOR SOILS WITH LIQUID LIMIT GREATER THAN 50

RESULTS OF 38 TESTS

PLATE #37

PLATE # 38



MOISTURE CONTENT vs. BLOW COUNT AT 0.8 FT PENETRATION

RESULTS OF 200 TESTS

KEUFFEL & ESSER CO., N. Y. NO. 359-81
Semi-Logarithmic, 4 Cycles X 70 to the inch
MADE IN U. S. A.

PLATE 38

APPENDIX A

Definition of Terms

1. Soil - This term includes all the loose or moderately cohesive earth materials such as gravels, sands, silts or clays or any of their mixtures.

2. Rock - The term rock is applied to natural beds or to large hard fragments of the original igneous sedimentary or metamorphic rocks.

3. Till - That part of the glacial drift deposited directly by ice without transportation or sorting by water. A wide range of grain size is typical of a till.

4. Clay - According to the definition commonly used in Engineering it is plasticity characteristics that define clay soils. In general soils which plot above the "A" line on the plasticity chart are defined as clays. On this basis the glacial tills found in the Edmonton area would be classified as clays from the point of view of soil mechanics.

5. Consolidation - The term applied to the process by which the increased stress applied to a clay is gradually transferred from the pore water to the soil skeleton with a corresponding decrease in volume of the soil.

6. Unconfined compressive strength - The maximum stress that a soil sample will carry when compressed in such a manner that during the compression it is free to expand laterally (Appendix B).

7. Swelling Pressure - The load, in the swelling test, (Appendix B) required to return a soil sample to its original volume after it has been allowed to swell under an initial token load.

8. Undisturbed Samples - Samples in which the material has been subjected to so little disturbance that it is suitable for all laboratory tests and thereby for approximate determination of the strength, consolidation and permeability characteristics and other physical properties of the material in situ.

APPENDIX B

1. Unconfined Compression Strength Tests

This test is run on specimens cut from undisturbed samples obtained in Shelby tubes. The specimens are either one inch in diameter, in which case three specimens are cut side by side from one Shelby tube sample, or three inches in diameter in which case the sample is merely extruded from the Shelby tube, cut to the required length and tested. In either case the length-diameter ratio of the sample tested is made approximately 2.5. The sample is loaded in a loading device using gravity loading. The load is applied by small known weights acting on a lever system to produce a known load on the specimen. Loads are applied in equal increments every 30 seconds and the corresponding deformation after 15 seconds is measured on an extensometer having a maximum travel of 1 inch. The maximum load carried by the specimen is taken as the unconfined compressive strength of the soil. The moisture content of each unconfined compressive strength sample at the end of the test is determined.

2. Combined Consolidation and Swelling Pressure Test

The consolidation test is run on specimens approximately $2\frac{1}{2}$ inches diameter by 1 inch high cut from Shelby tube samples. The test is carried out in a fixed ring consolidation apparatus similar to the one described by Lambe (Ref. 9) and following the procedure recommended by Lambe with the following exceptions.

APPENDIX B (cont'd)

i. For the initial load (usually 0.008 tons per square foot) water is allowed to reach the specimen from the bottom only.

ii. With the initial token load in place the soil is allowed to swell and the load is not increased until swelling is complete.

iii. After the soil has stopped swelling water is added to submerge the specimen and any further swelling is allowed to take place.

iv. After the submerged sample has stopped swelling the load is increased making each load increment approximately equal to the previous load on the specimen as described by Lambe.

v. The swelling pressure is taken as the load required to return the soil sample to its original volume. It can most easily be determined from the pressure-void ratio plot.

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